# Stormwater Site Plan

## Issaquah Evergreen Ford
## Issaquah, WA

**Prepared For:**
Evergreen Ford
1500 18th Ave.
Issaquah, WA 98027

**Prepared By:**
SCJ Alliance
8730 Tallon Lane NE, Suite 200
Lacey, WA 98516
360-352-1465

July 2019

SCJ ALLIANCE
CONSULTING SERVICES

# Stormwater Site Plan

*Project Information*

| | |
|---|---|
| Project: | **Issaquah Evergreen Ford** |
| Prepared for: | **Evergreen Ford**<br>1500 18th Ave.<br>Issaquah, WA 98027<br>Contact Name: Daniel Rowe<br>Contact Phone: 425-392-6900 |

*Reviewing Agency*

| | |
|---|---|
| Jurisdiction: | City of Issaquah |

*Project Representative*

| | |
|---|---|
| Prepared by: | **SCJ Alliance**<br>8730 Tallon Lane NE, Suite 200<br>Lacey, WA 98516<br>360.352.1465<br>scjalliance.com |
| Contact: | Tyrell Bradley, PE |
| Project Reference: | SCJ #1883.01<br>Path: N:\Projects\1883 Strotkamp Architects\1883.01 Evergreen Issaquah Ford\Phase 02 - Construction Documents\Design\Storm\Stormwater Site Plan\2019-xxxx Stormwater Site Plan.docx |

# PROJECT ENGINEER'S CERTIFICATION

I hereby certify that this Stormwater Site Plan for the Issaquah Evergreen Ford Dealership project has been prepared by me or under my supervision and meets the minimum standards of the City of Issaquah and normal standards of engineering practice. I hereby acknowledge and agree that the jurisdiction does not and will not assume liability for the sufficiency, suitability, or performance of drainage facilities designed by me.

7/10/2019

Prepared by: Mallory Dobbs, EIT — Date
Mallory.Dobbs@scjalliance.com
(360) 352-1465


7/10/2019

Approved by: Tyrell, Bradley, PE — Date
Tyrell.Bradley@scjalliance.com
(360) 352-1465

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# 1. PROJECT OVERVIEW

The following report was prepared for the Issaquah Evergreen Ford Dealership project in Issaquah, WA. This report was prepared to comply with the minimum technical standards and requirements that are set forth in the *2014 Department of Ecology Stormwater Management Manual for Western Washington (SWMMWW)* and the 2017 Stormwater Design Manual Addendum.

| | |
|---|---|
| **Project Proponent:** | Evergreen Ford |
| **Parcel Numbers:** | 2724069084, 2724069086 |
| **Total Parcel Area:** | 3.92 Acres |
| **Current Zoning:** | IC – Intensive Commercial |
| **Required Permits:** | Grading, Utility, Paving, Building, etc. |
| **Site Address:** | 6721 30$^{th}$ Ave. SE |
| **Section, Township, Range:** | Section 27, Township 24 N, Range 6 W |

The proposed Evergreen Ford site is located on two parcels that contain a total of 3.92 acres. The project is located on the south east corner of E Lake Sammamish Parkway SE and 229$^{th}$ Ave SE in Issaquah, WA. The proposed construction includes the 4-story ford dealership building/parking garage, as well as associated parking lot, utilities, frontage improvements, and stormwater improvements disturbing approximately 3.44 acres. Specifically, the proposed site improvements/construction activities for this project include the following:

- Site preparation, grading, and erosion control activities
- Construction of Ford dealership and parking garage
- Construction of parking lot
- Construction of off-site improvements
- Construction/installation of on-site water quality and flow control facilities
- Extension of available utilities (i.e., water, sewer, etc.)

A site vicinity map of the proposed project location is enclosed herein as **Appendix 1**. A worksheet for determining the number of Minimum Requirements for this project per the SWMMWW has been prepared and enclosed herein as **Appendix 2**. Per Table 1-1 from the City of Issaquah 2017 Stormwater Design Manual Addendum, the proposed project is a new development not located within the Central Issaquah Alternative Flow Control area and will created over 5,000 S.F. of new hard surfaces, therefore the project will trigger Minimum Requirements #1-9. Additionally, the pre-developed conditions must be modeled in forested.

| Table 1-1 PROJECT SCREENING FOR STORMWATER REVIEW | | | | | | | |
|---|---|---|---|---|---|---|---|
| | Screening Thresholds[a] | | | Minimum Requirements[a] | | | |
| Project Type[b] | Hard Surfaces | | Land Clearing | MR #1-5 | MR #6-9 | Stormwater Facility Target Surfaces[d] | Pre-Dev Cond. |
| **1. TESC Only** | <2000 SF new plus replaced hard surfaces | or | <7000 SF land disturbance | MR #2 – Construction Stormwater Pollution Prevention Plan | | | |
| **2. New Development** – *All projects*[c] | 2000-5000 SF new plus replaced hard surfaces | or | 7000-32,670 SF land disturbance | ✓ | | -- | -- |
| | >5000 SF new plus replaced hard surfaces | or | >32,670 SF land disturbance | ✓ | ✓ | New and replaced hard surfaces | Forested |
| **3a. Redevelopment** - *Value of proposed improvements is <50% of value of existing site improvements*[c] | 2000-5000 SF new plus replaced hard surfaces | or | 7000-32,670 SF land disturbance | ✓ | | -- | -- |
| | >5000 SF new plus replaced hard surfaces | or | >32,670 SF land disturbance | ✓ | ✓ | New hard surfaces only | Forested |
| **3b. Redevelopment** - *Value of proposed improvements is >50% of value of existing site improvements*[c] | 2000-5000 SF new plus replaced hard surfaces | or | 7000-32,670 SF land disturbance | ✓ | | -- | -- |
| | >5000 SF new plus replaced hard surfaces | or | >32,670 SF land disturbance | ✓ | ✓ | New and replaced hard surfaces | Forested |
| **4a. Transportation redevelopment** - *New hard surfaces add <50% to existing hard surfaces* | 2000-5000 SF new plus replaced hard surfaces | or | 7000-32,670 SF land disturbance | ✓ | | -- | -- |
| | >5000 SF new plus replaced hard surfaces | or | >32,670 SF land disturbance | ✓ | ✓ | New hard surfaces only | Forested |
| **4b. Transportation redevelopment** - *New hard surfaces add >50% to existing hard surfaces* | 2000-5000 SF new plus replaced hard surfaces | or | 7000-32,670 SF land disturbance | ✓ | | -- | -- |
| | >5000 SF new plus replaced hard surfaces | or | >32,670 SF land disturbance | ✓ | ✓ | New and replaced hard surfaces | Forested |
| **5. Central Issaquah Alternative Flow Control Area (see Figure 2-5)** – *All projects* | 2000-5000 SF new plus replaced hard surfaces | or | 7000-32,670 SF land disturbance | ✓ | | -- | -- |
| | >5000 SF new plus replaced hard surfaces | or | >32,670 SF land disturbance | ✓ | ✓ | New hard surfaces only | Existing |

**Figure 1: Project Screening for Stormwater Review**

## 1.1 SUMMARY OF COMPLIANCE ON-SITE

The stormwater design complies with the 9 minimum requirements as follows:

Minimum Requirement #1 – Preparation of Stormwater Site Plans – The Stormwater Site Plan is prepared per the 2014 SWMMWW.

Minimum Requirement #2 – Construction Stormwater Pollution Prevention – A pollution prevention plan will be completed and included with the stormwater site plan as Appendix 7 at the time of the civil permit submittal which will describe the 13 required elements. Further, an erosion control plan will be prepared and included as part of the engineering construction plan set in Appendix 4.

Minimum Requirement #3 – Source Control of Pollution – BMPs listed below are the minimum required for the site, additional BMPs not listed here may need to be implemented the meet the minimum requirements discussed in the 2014 SWMMWW.

- S411 BMPs for Landscaping and Lawn/Vegetation Management
- S417 BMPs for Maintenance of Stormwater Drainage and Treatment Systems
- S421 BMPs for Parking and Storage of Vehicles and Equipment
- S426 BMPs for Spills of Oil and Hazardous Substances

Minimum Requirement #4 – Preservation of Natural Drainage Systems and Outfalls – Currently, stormwater runoff within the parcels sheet flows into the two streams located on and adjacent to the parcels. The roadway frontage along 230$^{th}$ Ave. SE sheet flows into ditches located along the eastern parcel line. A portion of the stormwater runoff from 229$^{th}$ Ave. and 66$^{th}$ Street flows directly into the stream off of the roadway. Ultimately, all of the stormwater runoff is discharged into the streams and taken to the north. After construction, the proposed development will detain the stormwater runoff and release it at the predeveloped (forested condition) rates into the ditch. The stormwater runoff flows from the project site will decrease since the current condition of the site is pasture/lawn. The stormwater runoff from 229$^{th}$ Ave. and 66$^{th}$ Street will no longer sheet flow directly into the stream, it will flow into the proposed gutter and be collected by catch basins. The catch basins will collect the stormwater runoff and convey it into a rain garden facility located within the right-of-way.

Minimum Requirement #5 – On-site Stormwater Management – In accordance with Minimum Requirement #7, this project is not flow control exempt. Using Table I-2.5.1: On-Site Stormwater Management Requirements for Project Triggering Minimum Requirements #1-9, the proposed project is a new development not located in the UGA on a parcel smaller than 5 acres, therefore the project shall employ the On-Site Stormwater Management BMPs in accordance with the Low Impact Performance Standard or List #2. The project will demonstrate compliance with List #2, see below.

**Lawn and Landscaped Areas:**

- Per the 2014 SWMMWW manual, BMP T5.13: Post Construction Soil Quality and Depth will be utilized to the maximum extent practicable. See landscape plans for details.

**Roofs:**

- Full Dispersion (BMP T5.30) or Downspout Full Infiltration Systems (BMP T5.10A): Full dispersion is not feasible for this project site. Full dispersion requires that the site protects at least 65% of the site in a forest or native condition. For this reason alone this BMP is not feasible. In addition, the existing topography and stream locations combined with the site plan does not allow for the required native flow paths at the appropriate slopes (less than 15% away from the target surfaces). Full Infiltration Systems are also not feasible for the project site. Due to the high groundwater, a mounding analysis was conducted and the required minimum separation of 3 feet from the bottom of the facility to the high groundwater is not achievable. All of the stormwater runoff from the proposed site improvements will be collected, treated, detained, and released at the predeveloped rates into the adjacent ditch.

**Other Hard Surfaces:**

- Full Dispersion (BMP T5.30): Full dispersion is not feasible for this project site for the reasons mentioned above.
- Permeable Pavement (BMP T5.15): Based on the use of the site and the location of the parcel, both enhanced treatment and phosphorous treatment are required for the stormwater runoff prior to infiltration. A permeable pavement system would not allow for the stormwater runoff to be treated prior to infiltration into the soils.
- Bioretention (BMP T7.30): Bioretention is feasible for a portion of the proposed project. A portion of the stormwater runoff from the frontage improvements will be collected and conveyed to a bioretention facility located within the right-of-way.
- Sheet Flow Dispersion (BMP T5.12) or Concentrated Flow Dispersion (BMP T5.11): Sheet flow dispersion and concentrated flow dispersion are both not feasible for this project. The locations of the existing streams do not allow for the required native flow paths for the stormwater runoff coming off of the target surfaces. Additionally, the requirements that need to be met for Minimum Requirement #6 require that the stormwater runoff be collected and treated prior to infiltration into the soils, this would not be possible prior to dispersion.

Minimum Requirement #6 – Runoff Treatment – The proposed project will construct over 5,000 S.F. of pollution-generating impervious surface, therefore a stormwater treatment facility is required. The SWMMWW states that enhanced treatment is required for project sites that discharge directly to fresh waters or conveyance systems tributary to fresh water designated for aquatic life use or that have an existing aquatic life use; or use infiltration strictly for flow control – not treatment – and the discharge is within ¼ mile of a fresh water designated for aquatic life use. The proposed project will be discharging the stormwater runoff into a ditch that is tributary to a fish bearing stream and therefore enhanced treatment is required for all of the pollution-generating impervious surfaces. The proposed project will not be discharging directly into the stream and therefore phosphorous treatment is not required per Section 1.2.2.3 of the 2017 Stormwater Design Manual Addendum. However, it is important to note that the Modular Wetland systems do provide phosphorous treatment as well. At this time the proposed project is not considered a high-use site, therefore oil-control is also not required. Enhanced treatment for the pollution-generating impervious surfaces will be provided through two Modular-Wetland Systems and the bioretention soil mix located within the bioretention facility in the right-of-way.

Minimum Requirement #7 – Flow Control – The proposed project will construct over 10,000 S.F. of effective impervious surfaces and will not be discharging into flow control exempt waters per Appendix I-E of the SWMMWW, Flow Control-Exempt Surface Waters. Therefore, flow control is required for this project. The proposed project is split into two drainage basins, one for the on-site improvements and the other for the frontage improvements. The stormwater runoff will be detained using an underground CMP detention system. The stormwater runoff will then be pumped out of the detention system at the predeveloped rates into a flow spreader approximately 25 feet from the centerline of the ditch. Therefore, the existing drainage patterns of the ditch will not be altered with the proposed project.

Minimum Requirement #8 – Wetlands Protection – There are no wetlands on the project site nor does the project site does currently discharge into a wetland.

Minimum Requirement #9 – Operation and Maintenance – An operations and maintenance manual will be included and attached herein as Appendix 6 at the time of the civil permit submittal.

# 2. EXISTING CONDITIONS SUMMARY

## 2.1 EXISTING ON-SITE CONDITIONS

The subject site is +/- 3.92 acres in size. Topography within the property generally flat throughout the site except for the side slopes of the North Fork Issaquah Creek that runs through the northwest corner. In 2017, the Washington State Department of Transportation (WSDOT) conducted the N Fork Issaquah Creek Fish Passage project on this parcel. This project included the following:

- Re-routing the N Fork Issaquah Creek to the west underneath E Lake Sammamish Parkway, instead of straight through the project parcel
- Re-routing a smaller stream to flow directly west under E Lake Sammamish Parkway instead of south under the I-90 off ramp
- Associated improvements to the culverts and downstream flow paths to both streams

Associated with the streams, there are many critical areas on the project site. See Section 2.1.1 of this report for more information. See **Appendix 3** for a preliminary map outlining all the proposed project improvements.

Besides the stream relocation project mentioned above, the site has remained undeveloped since at least 1990. There are no known current drainage flow control facilities on the site. See the figures below.


Figure 2: Existing Conditions (1990)


Figure 3: Existing Conditions (2018)

### 2.1.1 Flood Hazard Zone

Flood Zones: The project parcel is located with Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) Panel No. 53033C0691H. According to the FIRM Map the project parcel contains Zone AE, Zone AH, and Zone X areas. Zone AE states that base flood elevations have been determined. Zone AH contains flood depths of 1 to 3 feet (usually areas of ponding); base flood elevations determined. The base flood elevation for this specific zone is 72. Zone X includes areas of 0.2% annual chance of flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance of flood. Per Issaquah Municipal Code (IMC) section 16.36.130, the proposed building must be constructed 1 foot above the base flood level. Therefore, the proposed finished floor elevation will be a minimum of 73. See **Appendix 8** for the FIRM Map.

Critical Area Recharge Area (CARA): According to the Critical Aquifer Recharge Area Classification Map (Exhibit C to Ordinance: CARA Map), the project parcel is located within the Class 1 – 1- & 5-year Wellhead Capture Zone. Per IMC 18.10.796, the City may require a groundwater monitoring plan and/or hydrogeologic critical area assessment report for new development projects. Per IMC 18.06.130, the proposed land use of an Automobile and Truck Sales/Dealership located in an intensive commercial zone and Class 1 CARA is not a prohibited or restricted use (IMC 18.06.130). Groundwater mounding proved that infiltration was not feasible for the project site. Therefore, all of the stormwater runoff from the project site will be collected, treated, detained, and pumped into the adjacent ditch at the predeveloped flow rates.

Streams and Stream Buffers: As mentioned above, the project parcel contains two streams with associated buffers. The N Fork Issaquah Creek is considered a Class 2 stream with salmonids. According to IMC 18.10.780, this stream is smaller than a Class 1 stream that flows year-round during periods of normal rainfall and all streams that are used by salmonids. The smaller stream to the south is considered a Class 4 stream. Per IMC 18.10.785, a Class 4 stream is a constructed or channelized stream, that is intermittent, not used by salmonids and do not provide salmonid habitat, and/or are not directly connected to a Class 1, 2, or 3 stream by an above ground channel. During the WSDOT project mentioned above, the stream buffer was reduced by 25% to create a 75' total buffer width. This buffer width has been added to the proposed project plans. The streams and stream buffers are graphically shown on the exhibit included in **Appendix 3**.

## 2.1.2 On-Site Soils Information

A geotechnical investigation was conducted by GeoEngineers in November, 2018. Eight test pits and five boring/monitoring wells were conducted to depths of approximately 5 to 81.5 feet. The surficial soils in the vicinity of the site are mapped as alluvial deposits, modified land, recessional outwash and advance outwash. Several stages of outwash and glacial deposition occurred along the Lake Sammamish area and along the outwash channels that carried glacial meltwater into glacial Lake Sammamish. The modified land in this area is typically fill placed to backfill gravel mining activities or to construct embankments for infrastructure. Subsurface soil and groundwater conditions encountered in the explorations were consistent with the geologic mapping. In general, GeoEngineers encountered a surficial layer of fill overlying a relatively thin layer of alluvium which increases in depth to the northwest. Medium dense to dense sand with variable silt, with an occasional layer of gravel with silt and sand underlies the alluvium (recessional outwash potentially transitioning to higher energy glaciofluvial deposits or transitional deposits). Groundwater was encountered during drilling and in the test pit excavations at a depth of 7 to 9 feet in all explorations. Groundwater was measuring as varying between a depth of 6.5 to 8 feet on January 14, 2019. A groundwater mounding analysis and report was completed by GeoEngineers in May of 2019. The mounding analysis concluded that even shallow infiltration facilities would not meet the required minimum separation from the bottom of the facility to the high groundwater. Therefore, infiltration was determined to be infeasible for the proposed project. See **Appendix 5** for the geotechnical reports.

# 3. OFFSITE ANALYSIS REPORT

## 3.1 Qualitative Upstream Analysis

Currently, stormwater runoff from 66th Street and 229th Avenue sheet flows directly from the roadway and into the stream. Off-site improvements will alter this flow path after construction. The off-site improvements along the south side of 66th Street and 229th Avenue include the construction of a sidewalk, planter strips, curb and gutter, and on-street parallel parking. Currently, the sidewalk ends at the intersection of East Lake Sammamish Parkway and 229th Ave. The proposed project will connect the sidewalk from East Lake Sammamish Parkway to the entrance of the proposed site. The stormwater runoff from the centerline of 229th Avenue and 66th Street currently sheet flows south/southeast directly into the stream buffer and into the stream. After construction, the stormwater runoff will flow along the proposed gutter line, into catch basins and conveyed into the bioretention facility located in the right-of-way. The stormwater runoff will infiltrate 100% in the bioretention facility. The frontage improvements to the east of the main entrance on 66th Street will flow along the gutter line around the corner and into 230th Avenue. Stormwater runoff from 230th Avenue currently sheet flows into ditches located on the east and west side of the roadway. After construction of the frontage improvements along 230th Ave., the stormwater runoff from the centerline to the west will be collected by catch basins and conveyed into the ditch to the south as it does today. This outfall will not be altered, and downstream conveyance systems are not anticipated to be adversely affected.

## 3.2 Qualitative Downstream Analysis

All of the stormwater runoff generated by the disturbed and developed area of the parcel will be detained in an underground detention facility. A pump system will pump the stormwater runoff at the predeveloped flow rates up into a flow spreader located outside of the Class 4 Stream buffer. The flow spreader will allow for the stormwater runoff to disperse, therefore not created a new outfall while also not scouring away the soil or plants nearby. There are no anticipated adverse effects to the downstream area of the project site. The proposed frontage improvements along 230th Avenue will not be constructing a significant amount of new impervious surface (<2,000 S.F.) and therefore no adverse effects to the downstream conveyance are anticipated at this time.

# 4. PERMANENT STORMWATER CONTROL PLAN

## 4.1 SUMMARY SECTION

The proposed project follows the development requirements stated in the 2014 SWMMWW and the 2017 Addendum to Stormwater Design Manual. Following Figure 2.4.1 (See **Appendix 2**), this project classifies as a new development that triggers all of the minimum requirements. The site does not have 35% or more of existing impervious coverage, and the project will add more than 5,000 S.F. of new impervious surfaces. See **Appendix 4** for the proposed stormwater facility locations and details. Table 1: Land Type Designations Existing vs. Proposed below illustrates the existing and proposed impervious and pervious areas of the disturbed areas (See **Appendix 3** for the basin map).

| LAND TYPE DESIGNATIONS | AREA (ACRES) | % OF TOTAL AREA |
|---|---|---|
| **Existing Areas** | **3.44** | **100** |
| Impervious | 0.50 | 14.53 |
| Pervious | 2.94 | 85.47 |
| **Proposed Areas** | **3.44** | **100** |
| **Basin 1** | **3.10** | **90.12** |
| Roof | 1.00 | 29.07 |
| Asphalt | 1.70 | 49.42 |
| Sidewalk | 0.15 | 4.36 |
| Landscape | 0.25 | 7.27 |
| **Basin 2** | **0.34** | **9.88** |
| Roof | 0.00 | 0 |
| Asphalt | 0.18 | 5.23 |
| Sidewalk | 0.09 | 2.62 |
| Landscape | 0.07 | 2.03 |

**Table 1: Land Type Designations Existing vs. Proposed**

### 4.1.1 Performance Standards and Goals

Following Figure 2.4.1 – Flow Chart for Determining Requirements for New Development, the project site triggers the use of Minimum Requirements #1-9. All of the stormwater runoff from the disturbed area of the project parcels will be infiltrated on-site. Enhanced treatment will be provided for all of the pollution-generating impervious surfaces through the use of Modular Wetland Systems and infiltration through bioretention soil mix.

### 4.1.2 Flow Control System

Flow control is required for the proposed development and will be provided through a bioretention facility, and underground detention facilities. The 2012 Western Washington Hydrology Model (WWHM) was used to size the flow control facilities so that they will meet Minimum Requirement #7. 100% of the stormwater runoff that is conveyed to the bioretention facility will infiltrate within the facility. It is important to note that the bioretention facility located within the right-of-way is not an underground injection well and will maintain a minimum of 5-feet of separation between the bottom of the facility and the groundwater. All of the stormwater runoff on-site will be collected, treated, and detained within an underground detention system. WWHM was used to size the detention system for the appropriate volume, and to provide the required release rates of the system. The drainage plan with the detention and conveyance layouts has been included as **Appendix 4**. See **Appendix 9** for the WWHM reports.

- Basin 1: One detention system made up of Contech CMP Pipe with 72,828 C.F. of live storage will detain and release the stormwater runoff from the entire basin at the predeveloped rates. This vault will be designed to meet all setback requirements from property lines and structures and will mainly be located within the drive aisle of the parking lot. A duplex pump system installed within the facility that will route the stormwater from the detention system to a flow spreader and be released at the predeveloped rates.
- Basin 2: A 1-foot deep bioretention facility with a bottom area of 1,000 S.F. will infiltrate 100% of the stormwater runoff from this basin. This bioretention facility will be located within the right-of-way to remain a publicly owned stormwater facility. All of the stormwater runoff within this basin is from public roadway improvements.

### 4.1.3 Water Quality System

Enhanced treatment will be provided for the proposed development through Modular Wetland Systems and a bioretention facility. The Modular Wetland Systems will precede the detention system and therefore are required to treat the flow rate at or below which 91% of the runoff volume, as estimated by WWHM. At this stage in design, it is assumed that the stormwater runoff from the sidewalk areas will flow across the asphalt parking areas, and therefore were included in the treatment facility sizing. The Modular Wetland Systems are equipped with an internal bypass and therefore can be sized using the off-line water quality flow rates. See below for the treatment facility sizes. See **Appendix 3** for the Treatment Basin Map exhibit. The drainage plan with the locations of the treatment facilities has been included as **Appendix 4**. See **Appendix 9** for the WWHM reports.

- Basin 1:
  - All of the basin area was used in treatment sizing, assuming all of the stormwater runoff will flow across the pollution generating impervious surface. The roof area of the parking garage area was including in these calculations.
  - Required Water Quality Treatment Flow = 0.1409 cfs
  - Modular Wetland Size = 6’x8’
- Basin 2:
  - 0.21 acres of PGIS (roof area will be directly tightlined to the infiltration facility and therefore does not require treatment)
  - Required Water Quality Treatment Flow = 0.2142cfs
  - Modular Wetland Size = 8’x8’
- Basin 3: Treatment for this basin will be provided through a bioretention facility located within the right-of-way. This bioretention facility has been sized to provide flow control for this basin and will infiltrate 100% of the stormwater runoff through the bioretention soil mix, and therefore meeting treatment requirements.

### 4.1.4 Conveyance System Analysis and Design

All stormwater conveyance systems will be sized to convey the 24-hour 25-year storm within the pipe. All proposed stormwater pipes are a minimum of 12” at a minimum slope of 0.25%.

# 5. CONSTRUCTION STORMWATER POLLUTION PREVENTION PLAN (C-SWPPP)

A SWPPP will be prepared and attached herein as **Appendix 7** at the time of the civil permit submittal.

# 6. SPECIAL REPORTS AND STUDIES

See **Appendix 5** for the geotechnical report. No other special reports or studies were required for this project.

# 7. OTHER PERMITS

Utility, paving, building, and grading permits may need to be secured prior to beginning construction activities. Coverage under Washington State Department of Ecology Phase II National Pollutant Discharge Elimination System Stormwater Permit will also need to be secured prior to beginning construction activities.

# 8. OPERATION AND MAINTENANCE MANUAL

The owner of the Evergreen Ford will be responsible in maintaining all stormwater facilities on-site. An operation and maintenance manual will be provided at the time of the civil permit submittal as **Appendix 6**.

**END OF STORMWATER SITE PLAN**

# APPENDIX 1
## SITE VICINITY MAP

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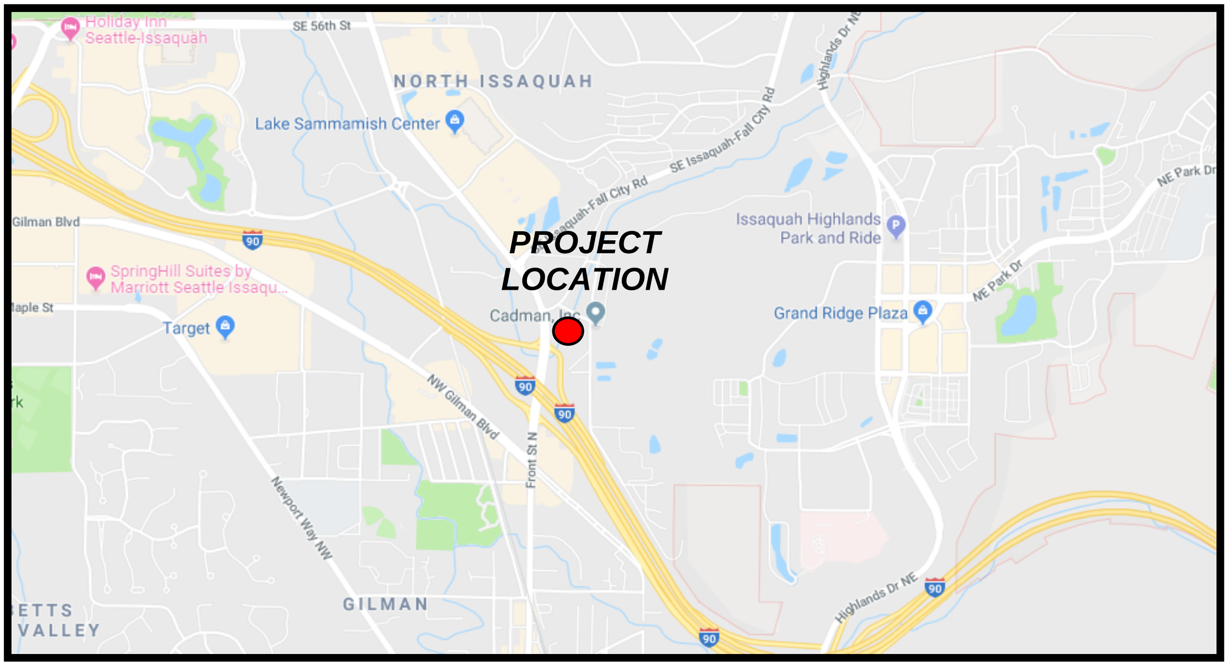
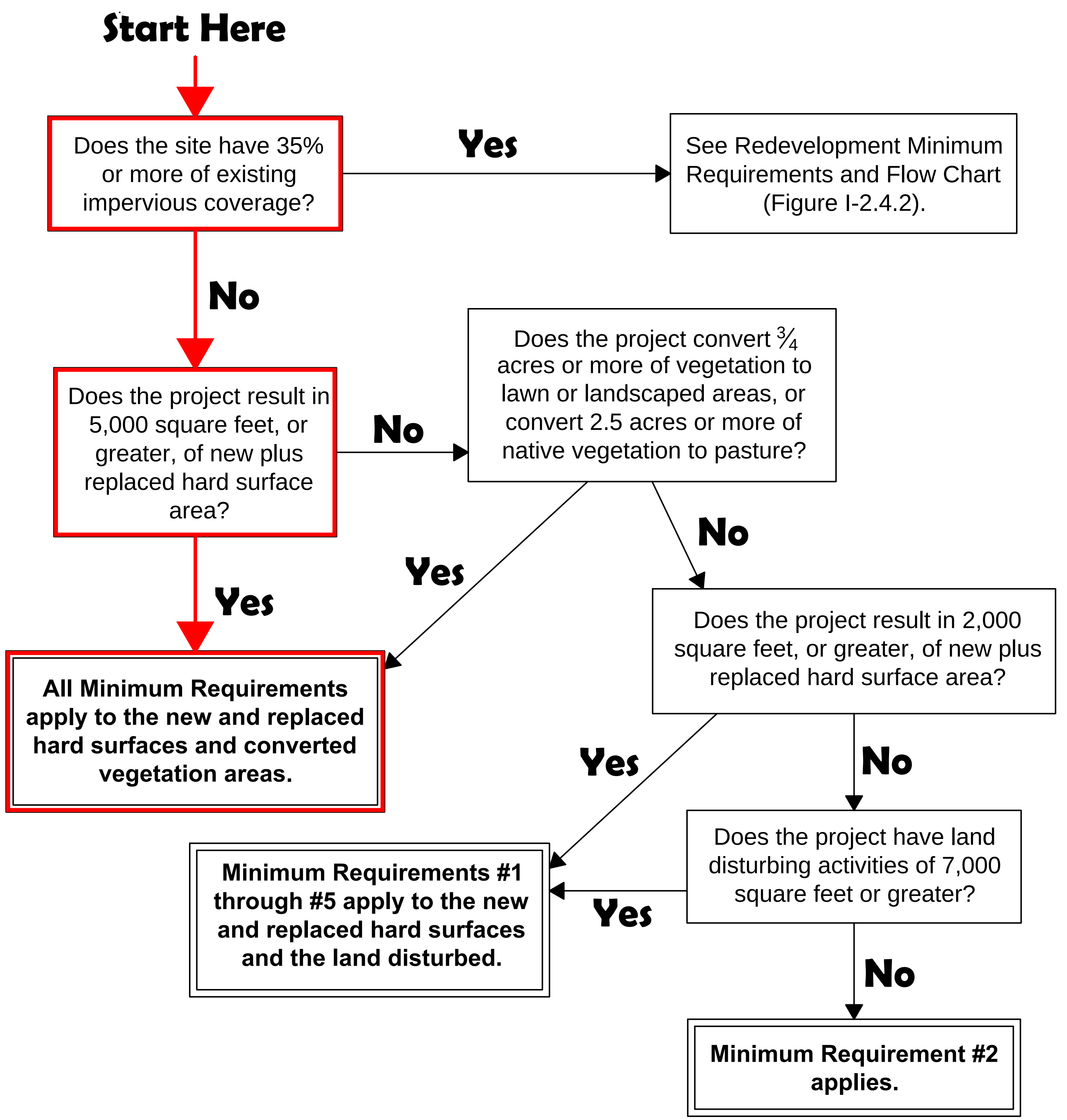



# APPENDIX 2
## DETERMINATION OF MINIMUM REQUIREMENTS WORKSHEET




DEPARTMENT OF ECOLOGY
State of Washington

Figure I-2.4.1
Flow Chart for Determining Requirements for New Development

Revised June 2015

# APPENDIX 3
## BASIN MAP EXHIBITS


BASIN 1 AREAS:

| | |
|---|---|
| ROOF: | 0.17 ACRES |
| ASPHALT: | 0.29 ACRES |
| SIDEWALK: | 0.04 ACRES |
| PERVIOUS: | 2.94 ACRES |
| TOTAL: | 3.44 ACRES |

Jun 17, 2019 1:54:41pm – User mallory.dobbs
N:\PROJECTS\1883 STROTKAMP ARCHITECTS\1883.01 EVERGREEN ISSAQUAH FORD\PHASE 01 – PRE-APPLICATION MEETS AND SITE PLAN REVIEW\CADD\EXHIBITS\1883.01 EX. BASIN MAP.DWG


SCJ ALLIANCE
CONSULTING SERVICES
8730 TALLON LANE NE, SUITE 200, LACEY, WA 98516
P: 360.352.1465 F: 360.352.1509
SCJALLIANCE.COM


HORIZONTAL SCALE: 1"=30'

DATE: JUNE, 2019

JOB No.: 1883.01

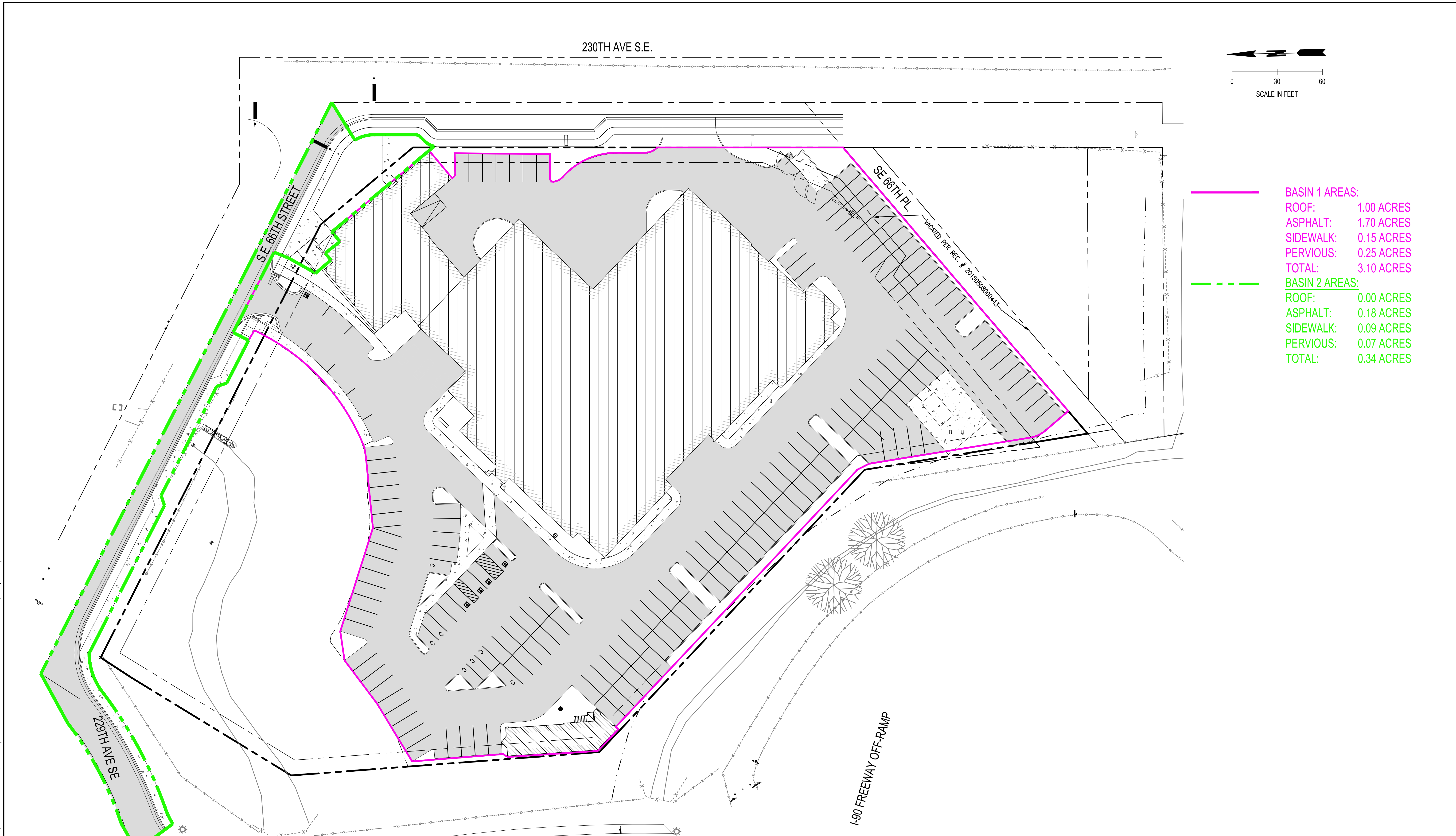
DRAWING FILE No.: 1883.01 Ex. Basin Map.dwg

EXISTING STORMWATER BASIN MAP

ISSAQUAH EVERGREEN FORD

EXHIBIT No: EX-01

SHEET No: 1

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SCJ ALLIANCE
CONSULTING SERVICES
8730 TALLON LANE NE, SUITE 200, LACEY, WA 98516
P: 360.352.1465 F: 360.352.1509
SCJALLIANCE.COM


HORIZONTAL SCALE: 1"=30'

DATE: JUNE, 2019

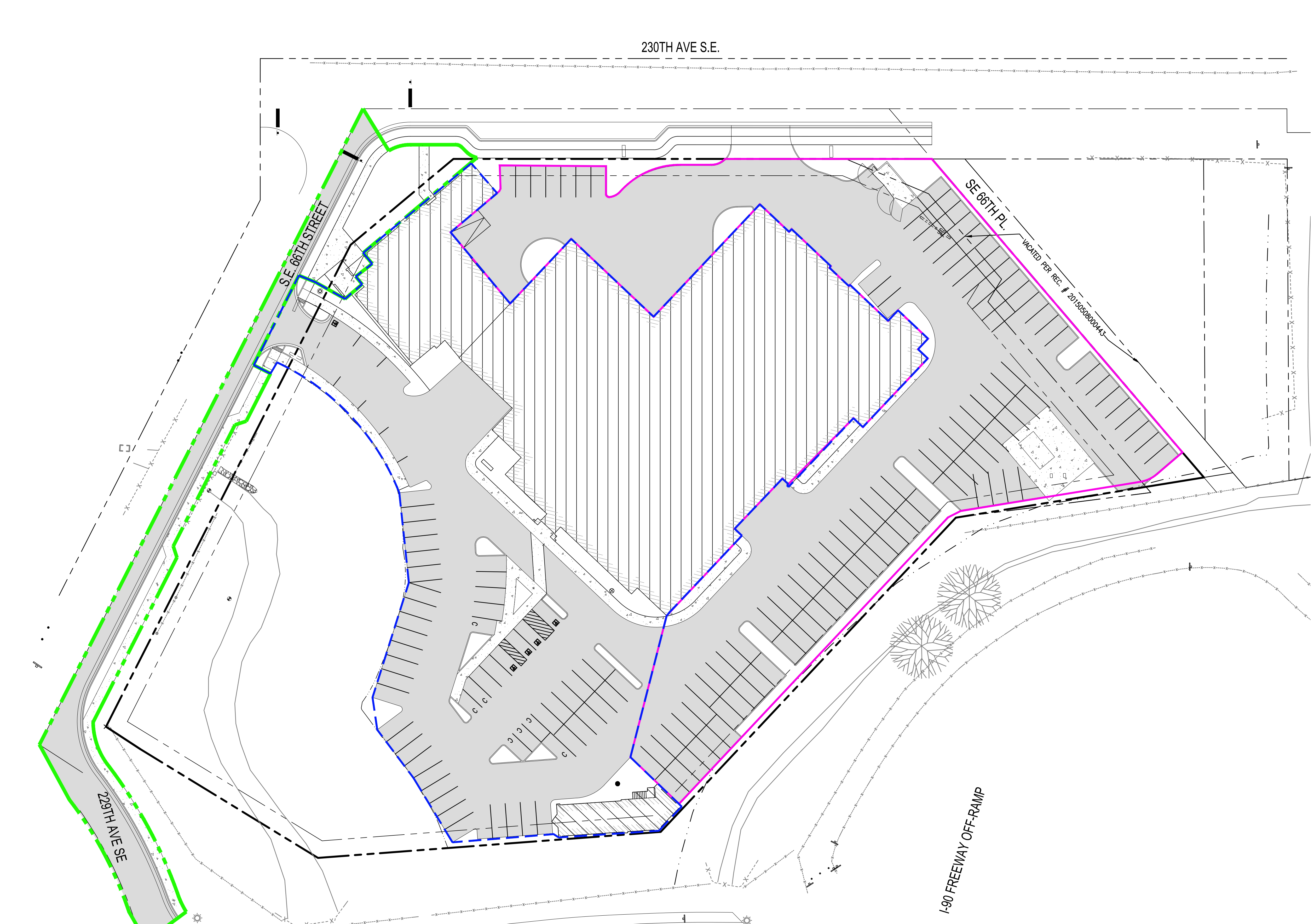
JOB No.: 1883.01

DRAWING FILE No.: 1883.01 Basin Map.dqg

# PROPOSED STORMWATER BASIN MAP
## ISSAQUAH EVERGREEN FORD

EXHIBIT No: EX-02

SHEET No: 2




BASIN 1 AREAS:

| | |
|---|---|
| ROOF: | 0.00 ACRES |
| ASPHALT: | 1.05 ACRES |
| SIDEWALK: | 0.08 ACRES |
| PERVIOUS: | 0.12 ACRES |
| TOTAL: | 1.25 ACRES |

BASIN 2 AREAS:

| | |
|---|---|
| ROOF: | 1.00 ACRES |
| ASPHALT: | 0.65 ACRES |
| SIDEWALK: | 0.07 ACRES |
| PERVIOUS: | 0.13 ACRES |
| TOTAL: | 1.85 ACRES |

BASIN 3 AREAS:

| | |
|---|---|
| ROOF: | 0.00 ACRES |
| ASPHALT: | 0.18 ACRES |
| SIDEWALK: | 0.09 ACRES |
| PERVIOUS: | 0.07 ACRES |
| TOTAL: | 0.34 ACRES |

SCJ ALLIANCE
CONSULTING SERVICES
8730 TALLON LANE NE, SUITE 200, LACEY, WA 98516
P: 360.352.1465 F: 360.352.1509
SCJALLIANCE.COM

HORIZONTAL SCALE: 1"=30'
DATE: JUNE, 2019
JOB No.: 1883.01
DRAWING FILE No.: 1883.01 Tr. Basin Map.dwg

TREATMENT BASIN MAP
ISSAQUAH EVERGREEN FORD

EXHIBIT No: EX-03
SHEET No: 3

Jun 17, 2019 2:51:56pm – User mallory.dobbs
N:\PROJECTS\1883 STROTKAMP ARCHITECTS\1883.01 EVERGREEN ISSAQUAH FORD\PHASE 01 – PRE-APPLICATION MEETS AND SITE PLAN REVIEW\CADD\EXHIBITS\1883.01 TR. BASIN MAP.DWG

# APPENDIX 4
## PRELIMINARY CONSTRUCTION PLANS

# APPENDIX 5

## GEOTECHNICAL REPORT

## Geotechnical Engineering Services

Evergreen Ford Lincoln
22909 SE 66$^{th}$ Street
Issaquah, Washington

*for*

**Strotkamp Associates and**
**Evergreen Ford Lincoln**

January 18, 2019

GeoEngineers
Earth Science + Technology

# Geotechnical Engineering Services

Evergreen Ford Lincoln
22909 SE 66th Street
Issaquah, Washington

*for*
**Strotkamp Associates and**
**Evergreen Ford Lincoln**

January 18, 2019

GeoEngineers
17425 NE Union Hill Road, Suite 250
Redmond, Washington 98052
425.861.6000

# Geotechnical Engineering Services

## Evergreen Ford Lincoln
## 22909 SE 66th Street
## Issaquah, Washington

**File No. 23589-001-00**

**January 18, 2019**

Prepared for:

Evergreen Ford Lincoln
c/o Strotkamp Architects
P.O. Box 501
Burlington, Washington 98233

Attention: Tom Strotkamp and David Estes

Prepared by:

GeoEngineers, Inc.
17425 NE Union Hill Road, Suite 250
Redmond, Washington 98052
425.861.6000

Nancy L. Tochko, PE
Senior Geotechnical Engineer

DEBRA C. OVERBAY
STATE OF WASHINGTON
29247
REGISTERED
PROFESSIONAL ENGINEER
1/18/19

Debra C. Overbay, PE
Associate

DTM:MSH:NLT:DCO:nld

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## APPENDICES

## INTRODUCTION AND PROJECT UNDERSTANDING

This report presents the results of our geotechnical engineering services in support of the new dealership building and parking lot for Evergreen Ford Lincoln located at 22909 SE 66th Street in Issaquah, Washington. The property is bounded by East Lake Sammamish Parkway SE on the west, 229th Avenue SE and SE 66th Street on the north, 230th Avenue SE on the east, and SE 66th Place and the I-90 off-ramp on the south. The project site is shown relative to surrounding physical features on the Vicinity Map (Figure 1) and Site Plan (Figure 2).

We understand the site is approximately 3½ acres in size, although the northwest corner of the site is occupied by a new creek channel created for the North Fork of Issaquah Creek. The creek channel was originally aligned on the east side of the kennel prior to it being moved in 2017. A second new creek channel borders the southern site boundary along the Issaquah-Preston Trail. The remaining site area is currently vacant, with the exception of an old dog kennel situated in the west and a cell tower located in the south.

We understand that the proposed new dealership will include a four- to five-story, at-grade concrete structure that forms a broad "L" shape measuring approximately 200 feet in length along the northwest and 250 feet along the southwest. The interior dimension will range from about 100 to 150 feet in width. The ground floor will be occupied by sales, service, and associated support facilities and the upper three to four levels will be parking. The development will also include a one-story steel frame building in the north at the intersection of SE 66th Street and 230th Avenue SE, with a walkway to the larger L-shaped concrete structure. The site layout is shown in Figure 2.

Heavy column loads are anticipated due to the concrete structure and upper decks of car loading. Preliminary column loads from PSM Consulting Engineers, the project structural engineer, range from about 350 to 900 kips. We understand the floor load on the ground floor (sales and office) will be on the order of 150 pounds per square foot (psf). Deep foundations will be required to support the structure. The ground floor can be supported at grade provided some damage is acceptable resulting from liquefaction settlement for the design seismic event.

The purpose of this study is to review existing geotechnical information and to complete subsurface explorations at the project site as a basis for providing geotechnical engineering recommendations for design. Our specific scope of services includes:

- reviewing previous explorations completed in the vicinity of the site;
- completing five borings and installing shallow monitoring wells in three of the borings;
- completing eight test pits across the site to better define the characteristics of the near-surface soils and potential compressible deposits;
- providing geotechnical foundation recommendations;
- performing analyses for seismic design, building foundation and floor slab support;
- evaluating infiltration feasibility and provide preliminary infiltration rates based on grain size analyses; and
- preparing this Geotechnical Engineering Design Report.

## FIELD EXPLORATIONS

### Previous Explorations

GeoEngineers reviewed the logs of explorations completed by others as part of previous studies in the vicinity of the project site. One of the previous borings, B-1, is located on the southern site border from a 1997 project, “Proposed AT&T Tower - Issaquah” by AGRA Earth and Environmental dated February 27, 1997. The location of this boring is shown in Figure 2 and the log is presented in Appendix B, Previous Explorations.

### Field Explorations

Subsurface conditions at the site were evaluated by reviewing previous explorations in the immediate vicinity, and by completing eight test pits and five boring/monitoring wells to depths of approximately 5 to 81½ feet below existing ground surface (bgs). The explorations were completed between October 31 and November 2, 2018. The approximate locations of the explorations are shown in Figure 2. A detailed description of the field exploration program and logs of the explorations are presented in Appendix A, Field Explorations and Laboratory Testing.

### Laboratory Testing

Soil samples obtained from the test pits and borings were transported to our Redmond geotechnical laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soil types encountered. Representative samples were selected for laboratory testing consisting of moisture content tests, percent fines, and sieve analyses.

## SITE CONDITIONS

### Geology

Published geologic information for the project vicinity includes “The Geologic Map of the Issaquah 7.5’ Quadrangle, King County, Washington (Booth, D.B., and Minard, J.P. 1992). The surficial soils in the vicinity of the site are mapped as alluvial deposits, modified land, recessional outwash and advance outwash. Several stages of outwash and glacial deposition occurred along the Lake Sammamish area and along the outwash channels that carried glacial meltwater into glacial Lake Sammamish. Ice contact deposits and transitional deposits are also mapped along the borders of the lake.

The modified land in this area is typically fill placed to backfill gravel mining activities or to construct embankments for infrastructure. Recessional deposits are mapped along the valley wall, below an upland till cap, and below the alluvial deposits.

The alluvial deposits generally consist of interbedded layers of loose/soft soil ranging from sand with variable silt content, to silt and gravel and can contain occasional layers of organic silt/peat. Recessional outwash deposits underlying the alluvium and mapped east of the site mainly consist of medium dense stratified sand and gravel, with some zones of silty sand and silt. Advance glacial and glaciofluvial deposits underlying the recessional outwash deposits mainly consist of dense to very dense sand and gravel with varying amounts of silt.

## Surface Conditions

The site is bounded by industrial property to the east, residential and commercial property to the north, by East Lake Sammamish Parkway SE to the west, and by Interstate 90 to the south. The site is relatively flat, with a metal frame deteriorated dog kennel on the east side of the site and a cell tower in the south corner of the site. The site has recently been used for some stockpiled soils, and was recently regraded. Most of the site is covered in newly planted grass and occasional trees. Newly planted landscaped buffers are present along each new stream channel. Above ground high-voltage transmission lines cross the southeast corner of the site near the cell tower.

## Subsurface Conditions

Subsurface soil and groundwater conditions encountered in the explorations are consistent with the geologic mapping. In general, we encountered a surficial layer of fill overlying a relatively thin layer of alluvium which increases in depth to the northwest. Medium dense to dense sand with variable silt, with an occasional layer of gravel with silt and sand underlies the alluvium (recessional outwash potentially transitioning to higher energy glaciofluvial deposits or transitional deposits). Soils encountered in our explorations are described in more detail below.

Monitoring wells MW-1 and MW-2 were located in the southeast portion of the proposed building footprint and encountered medium dense silty sand fill with variable gravel content to a depth of about 8 feet below existing site grade. Soft silt and very loose to medium dense silty sand were encountered below the fill. The sand becomes dense to very dense below a depth of about 18 feet. Monitoring well MW-1 encountered dense gravel from a depth of about 19 to 24 feet, and below a depth of 30 feet. The borings were terminated in the sand and gravel at a depth of 31½ feet.

Monitoring well MW-3 and boring B-1 were located toward the northern end of the proposed building. Loose to medium dense silty sand fill was encountered to a depth of about 5 to 8 feet bgs. A layer of loose to very loose silty sand was encountered below the fill in MW-3 to a depth of about 13 feet. Medium dense silty gravel underlies the fill in boring B-1. Loose to dense sand with variable silt and gravel was encountered at depth in both explorations. Dense to very dense sand and gravel deposits were encountered at a depth of 40 to 45 feet. The explorations were terminated in dense sand at a depth between 46 and 52 feet.

Boring B-2 was located in the southeast portion of the proposed building footprint. Approximately 8 feet of loose surficial silty sand fill was also encountered in this boring. Medium dense sand and gravel underlies the fill to a depth of about 18 feet where an approximate 3-foot thickness of organic silt was encountered. Loose to medium dense sand and gravel was encountered below the organic silt to a depth of about 43 feet. An approximate 10-foot layer of dense gravel was encountered between a depth of 43 and 53 feet. Interlayered medium dense to dense sand with variable silt was encountered below this depth to the 80-foot depth explored.

Similar subsurface soil conditions were encountered in the test pits consisting primarily of loose to medium dense silty sand fill with variable gravel. Two test pits, TP-3 and TP-4 encountered a 1- to 1½-foot layer of soft organic silt at a depth of 4 to 5 feet, and test pit TP-7 encountered an approximate 3-foot thickness of soft peat.

Groundwater was encountered during drilling and in the test pit excavations at a depth of 7 to 9 feet in all the explorations. Groundwater was measured as varying between a depth of 6.5 to about 8 feet on

January 14, 2019; the measurements are presented on the respective monitoring well logs. Groundwater conditions should be expected to fluctuate as a function of season, precipitation, and fluctuations of the North Fork Issaquah Creek.

## CONCLUSIONS AND RECOMMENDATIONS

We conclude that the site is suitable for constructing the proposed building on deep foundations to support heavy structural loads and to mitigate for settlement due to liquefaction. The site is underlain by liquefiable soils and could experience settlement on the order of 2 to 6 inches during the design seismic event over the majority of the site. Greater liquefaction settlement in the range of 8 to 10 inches is estimated within the west side of the site. Based on the explorations completed to date, deep foundations extending to a depth of 30 to 40 feet in the east, 50 to 55 feet in the north and up to 90 feet in the west corner will support heavy column loads and extend below liquefiable soil layers. Additional explorations should be completed during final design to refine required embedment of deep foundations and confirm liquefiable soil depths.

The first floor should be designed as a structural slab due to the estimated range of liquefaction settlement across the site. Light foundation loads supporting other site facilities can be considered for shallow foundation support provided settlement due to liquefaction is acceptable.

Surficial soils at the site consist mostly of moisture sensitive silty sand fill. Based on the results of the laboratory tests, the on-site soils will likely not be re-usable as structural fill without significant moisture conditioning (aeration). Excavation and replacement of portions of the on-site soils should be anticipated to construct the recommended zone of structural fill beneath the first floor slab and subgrade for the surrounding parking area. Detailed geotechnical recommendations for foundation support and other aspects of project development are presented in the following sections.

### Earthquake Engineering

#### 2015 IBC Design Parameters

Based on the subsurface soils encountered in the explorations completed to date, the north and east portions of the proposed building area are underlain by soils classified as Site Class D, and soils encountered in the west corner of the building are classified as Site Class E. We recommend the use of the following 2015 International Building Code (IBC) parameters for soil profile type, short period spectral response acceleration ($S_S$), 1-second period spectral response acceleration ($S_1$) and seismic coefficients ($F_A$ and $F_V$) for the project site.

**TABLE 1. 2015 IBC DESIGN PARAMETERS**

| 2015 IBC Parameter | Recommended Value | |
|---|---|---|
| | Site Class D | Site Class E |
| Short Period Spectral Response Acceleration, $S_S$ (percent g) | 131 | 131 |
| 1-Second Period Spectral Response Acceleration, $S_1$ (percent g) | 49 | 49 |
| Seismic Coefficient, $F_A$ | 1 | 0.9 |
| Seismic Coefficient, $F_V$ | 1.51 | 2.4 |
| Peak Ground Acceleration (percent g) | 53 | 48 |

### Surface Faults

The site is more than 2 miles south of the Seattle Fault Zone and therefore it is our opinion the risk of surface fault rupture is low.

### Liquefaction

Liquefaction refers to the condition when vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength in the deposit of soil so affected. In general, soils that are susceptible to liquefaction include very loose to medium dense clean to silty sands and some silts that are below the water table. Liquefaction usually results in ground settlement and loss of bearing capacity, resulting in settlement of structures that are supported on foundations that are constructed within or above the liquefied soils.

We evaluated the liquefaction potential based on the current and previous explorations using the Simplified Procedure (Youd and Idriss 2001). The Simplified Procedure is based on comparing the cyclic resistance ratio (CRR) of a soil layer (the cyclic shear stress required to cause liquefaction) to the cyclic stress ratio (CSR) induced by an earthquake. The factor of safety against liquefaction is determined by dividing the CRR by the CSR. Liquefaction hazards, including settlement and related effects, were evaluated when the factor of safety against liquefaction was calculated as less than 1.0.

Based on our analysis using the 2015 IBC seismic event (peak horizontal acceleration of 0.53g), it is our opinion there is a moderate to high risk of liquefaction within the upper sand deposits as well as the silt. We estimate that the factor of safety is less than 1.0 during the design-level earthquake for a 10- to 25-foot total thickness of soil in the north, east and south, and up to an approximate 50-foot thickness in the west corner of the proposed building footprint.

The magnitude of liquefaction-induced ground settlement was computed using the Youd and Idriss (2001) simplified approach described previously. Reconsolidation settlement (volumetric strain) is estimated as a function of the factor of safety of liquefaction triggering (serving as a proxy for the maximum accumulated shear strain). Liquefaction-induced ground settlement of the potentially liquefiable zones across the building footprint is estimated to range from 2 inches (in the southeast in the vicinity of monitoring wells MW-1 and MW-2) to as much as 10 inches in the vicinity of boring B-2 for a design-level earthquake. Table 2 below summarizes the range of estimated liquefaction-induced settlement based on the conditions encountered in the explorations.

**TABLE 2. ESTIMATED LIQUEFACTION-INDUCED GROUND SETTLEMENT**

| Boring | Estimated Ground Surface Settlement[1] (inches) |
|---|---|
| B-1 | 4 to 6 |
| B-2 | 8 to 10 |
| MW-1 | 2 |
| MW-2 | 2 |
| MW-3 | 4 to 6 |

Note:

[1] Additional explorations should be completed during final design to better delineate potential deep liquefaction zones and optimize the building foundation

Lesser amounts of settlement from liquefaction could be experienced after an earthquake with a magnitude less than the design-level earthquake. The magnitude of liquefaction-induced ground settlement will vary as a function of the characteristics of the earthquake (earthquake magnitude, location, duration and intensity) and the soil and groundwater conditions.

## Pile Foundations

Based on the presence of potentially liquefiable soils in the upper 20 to 75 feet of the site, and the heavy column loading ranging from about 350 to 900 kips, we recommend that the proposed building be supported on deep foundations. Augercast piles are a common pile foundation in the northwest and typically offer the most economical foundation for heavy column loads. Recommended capacities for 18- and 24-inch-diameter augercast piles are provided below.

### Axial Capacity

Table 3 below presents the ultimate pile axial capacities for 18- and 24-inch-diameter piles with a minimum embedment depth of 30 feet in the vicinity of MW-1 and MW-2, 50 feet in the vicinity of boring B-1, and 90 feet at the location of boring B-2. We recommend additional explorations be completed in the footprint prior to contractor bidding to refine the pile depths across the footprint. These ultimate capacities include the down drag force induced by liquefiable soils. A factor of safety of 3 should be used to obtain allowable pile capacities. In addition to the downward compressive load from the seismic structural load, the soil down drag load presented below would need to be included in the structural design analysis.

**TABLE 3. ULTIMATE AXIAL PILE CAPACITIES**

| Pile | Embedment into Dense to Very Dense Sand and Gravel (feet) | Typical Pile Length[1] (feet) | | | Downward Capacity[2] (kips) | Uplift Capacity (kips) | Down-drag Force (kips) |
|---|---|---|---|---|---|---|---|
| | | MW-1, MW-2 | B-1 | B-2 | | | |
| 18-inch-diameter augercast | 10 | 30 | 50 | 90 | 390 | 50 | 32 |
| | 15 | 35 | 55 | 95 | 425 | 85 | |
| | 20 | 40 | 60 | 100 | 460 | 120 | |
| 24-inch-diameter augercast | 10 | -- | 50 | 90 | 740 | 63 | 43 |
| | 15 | 35 | 55 | 95 | 790 | 110 | |

Notes:

[1] Additional explorations should be completed during final design to refine pile embedment depths.

[2] A factor of safety of 3 should be used to obtain allowable static pile capacity.

### Lateral Capacity

Lateral loads can be resisted by passive soil pressure on the vertical piles and by the passive soil pressures on the pile cap. Due to the potential separation between the pile-supported foundation components and the underlying soil from settlement, base friction along the bottom of the pile cap should not be included in calculations for lateral capacity because full contact with the underlying soil cannot be assured.

We completed lateral pile capacity analyses for 18- and 24-inch diameter augercast piles using the computer software program LPILE 2016 produced by Ensoft, Inc. The analyses were completed for both a non-liquefied (static) and liquefied (seismic) soil profile.

Our LPILE analysis results are presented in Figures 3 through 14 as described in the table below. The depths shown on the figures are measured from the bottom of the pile cap; we do not anticipate these results will change significantly with variations in the top of pile elevation.

**TABLE 4. LATERAL PILE ANALYSES RESULTS**

| Figures | Results |
|---|---|
| Figures 3, 4 and 5 | 18-inch diameter augercast piles, Boring MW-2<br>Fixed head deflection, moment and shear diagrams for static and liquefied conditions |
| Figures 6, 7 and 8 | 18-inch diameter augercast piles, Boring B-2<br>Fixed head deflection, moment and shear diagrams for static and liquefied conditions |
| Figures 9, 10 and 11 | 24-inch diameter augercast piles, Boring MW-2<br>Fixed head deflection, moment and shear diagrams for static and liquefied conditions |
| Figures 12, 13 and 14 | 24-inch diameter augercast piles, Boring B-2<br>Fixed head deflection, moment and shear diagrams for static and liquefied conditions |

The results presented in Figures 5 through 14 are for single piles. Piles spaced closer than five pile diameters apart will experience group effects that will result in a lower lateral load capacity for trailing rows of piles with respect to leading rows of piles for an equivalent deflection. We recommend that the lateral load capacity for trailing piles in a pile group spaced less than five pile diameters apart be reduced in accordance with the factors in the table below per American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications Section 10.7.2.4.

**TABLE 5. PILE P-MULTIPLIERS, $P_M$, FOR MULTIPLE ROW SHADING**

| Pile Spacing[1] (in terms of pile diameter) | P-Multipliers, $P_m$[2] | | |
|---|---|---|---|
| | Row 1 | Row 2 | Row 3 and higher[3] |
| 3D | 0.8 | 0.4 | 0.3 |
| 5D | 1.0 | 0.85 | 0.7 |

Notes:

[1] The P-multipliers in the table above are a function of the center to center spacing of piles in the group in the direction of loading expressed in multiples of the pile diameter, D.

[2] The values of $P_m$ were developed for vertical piles only.

[3] The P-multipliers are dependent on the pile spacing and the row number in the direction of the loading. To establish values of Pm for other pile spacing values, interpolation between values should be conducted.

Resistance to lateral loads can also be developed by passive pressure on the face of pile caps and other below-grade foundation elements. The allowable passive resistance on the face of grade beams, pile caps, or other embedded foundation elements may be computed using an equivalent fluid density of 250 pounds per cubic foot (pcf) (triangular distribution) if these elements are cast in direct contact with undisturbed on-site soils. Alternatively, passive pressures may be computed using an equivalent fluid density of 350 pcf if all soil extending out from the face of the foundation element for a distance at least equal to two and one-half times the depth of the element consists of structural fill compacted to at least 95 percent of maximum dry density (MDD) (ASTM D-1557). This passive resistance value includes a factor of safety of

1.5 and a minimum lateral deflection of 1 inch to fully develop the passive resistance. Deflections less than 1 inch will not fully mobilize the passive resistance and can be linearly interpolated from the resistance at 1 inch.

#### Pile Installation

Augercast piles should be installed to the recommended penetrations using a continuous-flight, hollow-stem auger. The pile grout is pumped under pressure through the hollow stem as the auger is slowly withdrawn. Reinforcing steel for bending and uplift is placed in the fresh grout column immediately after withdrawal of the auger.

We recommend that the augercast piles be installed by a contractor experienced in their placement and using suitable equipment. Grout pumps should be fitted with a volume-measuring device and pressure gauge so that the volume of grout placed in each pile and the pressure head can be readily determined. While grouting, the rate of auger withdrawal should be controlled such that the rate is uniform and the volume of grout pumped is equivalent to at least 115 percent of the theoretical hole volume. A minimum grout line pressure of 100 pounds per square inch (psi) should be maintained while grouting. We recommend that there be a waiting period of at least eight hours between installation of piles spaced closer than 8 feet center-to-center, in order to avoid disturbance of concrete undergoing curing in a previously cast pile. This is particularly important for the anticipated depth and the loose soil consistency at the site. These materials can sometimes experience a “blow out” from grout pressures during augercast pile installation.

It should be noted that the recommended pile tip elevations and capacities presented above are based on assumed uniformity of soil conditions between the explorations. Obstructions could be encountered within the fill soils during installation such that new pile locations may need to be selected and/or pile capacities may need to be reevaluated. There may be unexpected variations in the depth to, and characteristics of, the supporting soils across the site. In addition, no direct information regarding the capacity of augercast piles (e.g., driving resistance data) is obtained while this type of pile is being installed. Therefore, it is particularly important that the installation of augercast piles be carefully monitored by a representative from our firm who will work under the direct supervision of an experienced engineer familiar with the conditions at this site.

### Floor Slab

As discussed previously, we estimate that potential settlements from soil liquefaction during a design earthquake event could vary significantly across the site. Because the estimated settlements are not tolerable, a structural floor slab is recommended. We recommend the floor slab be underlain by a minimum 4-inch-thick capillary break layer. to provide uniform support and drainage. Gradation recommendations for the capillary break are presented in the “Earthwork” section below.

If water vapor migration through the slabs is objectionable, such as in occupied spaces or areas where adhesives are used to anchor carpet or tile to the slab, the capillary break material should be covered with a commercial moisture vapor retarder (10-mil minimum thickness with lapped and sealed seams). The moisture vapor retarder should be constructed in accordance with the American Concrete Institute (ACI 302.1R) and placed over the capillary break layer. The contractor should be made responsible for maintaining the integrity of the vapor barrier during construction.

A waterproofing product designed for this purpose may be used in lieu of the capillary break material and vapor retarder if a more robust level of protection is desired.

## Ground Improvement

### Methods and Design Considerations

Ground improvement can be considered to mitigate liquefaction and provide increased bearing pressures for shallow footings. However, the depth and thickness of liquefiable soils vary significantly across the building footprint, and extend up to a depth of about 75 feet in boring B-2. Additional explorations should be completed to verify the extent and thickness of liquefiable zones during final design. The ground improvement system should be designed so that abrupt differential settlements do not occur along the transition line between differing thicknesses of liquefiable soil, and between improved ground and non-improved ground. As such, ground improvement may not be practical in areas of deep liquefiable soils.

Ground improvement options may include rigid inclusions, aggregate piers, and driven timber piles to mitigate liquefication and provide increased bearing for shallow foundations. The ground improvement elements should be installed in a grid pattern beneath footings, and also at regular intervals beneath the ground floor slab, as needed to limit slab settlements.

Rigid inclusions are unreinforced low strength concrete elements that transfer foundation loads through weak soils down to underlying competent soils. These are typically installed using a bottom-feed mandrel that is vibrated down to the bearing soils. Granular bearing soils are densified by displacement. Low strength concrete is pumped through the mandrel, which opens at the bottom as it is raised. The mandrel is extracted while a positive concrete pressure is maintained.

Rammed aggregate piers consist of holes created by driving/vibrating a mandrel which are then filled with densely compacted crushed rock. The holes are advanced down to suitable bearing soils. The crushed rock is placed in the hole in lifts of about 12 inches in thickness as the mandrel is withdrawn and compacted using a high energy hydraulic ram. Grout can be added to the portion of the crushed rock column extending through the peat in order to provide higher lateral stiffness and therefore a higher vertical load capacity and smaller foundation settlements.

Each of these methods involve displacing rather than replacing the existing soil. Accordingly, the resulting composite soil mass has improved strength, lower compressibility, and low liquefaction potential. Also, foundation loads are transferred to the underlying competent bearing soils.

The ground improvement systems would be completed on a grid pattern, where necessary, to transfer the foundation loading to the bearing soils. The type of ground improvement technique should be reviewed with the project team to identify constructability issues, provide a range of cost, and to establish the allowable bearing that can be achieved using the method selected.

A contractor specializing in ground improvement methods should develop a performance-based design that will meet the support and settlement criteria specified by the project structural engineer. We recommend that we be retained to review the proposed ground improvement program.

### Construction Considerations

Installation of ground improvement elements may encounter seepage or heaving conditions due to the high groundwater levels present at the site, and zones of medium dense to dense gravel. Measures should be taken to prevent sloughing, caving, heaving or running of soil into the holes. Casing or other techniques may be necessary to stabilize the holes. Also, concrete and asphalt pieces, debris, cobbles or boulders may be encountered during installation of the ground improvement elements.

Each of the ground improvement methods will generate vibrations during installation. These vibrations are not expected to adversely affect nearby off-site structures. However, it is likely that the vibrations will be felt by people within a limited area in and adjacent to the site.

GeoEngineers should observe and document the installation of the selected ground improvement method to verify conformance with the design assumptions and recommendations.

In our experience, building foundations bearing on a crushed rock pad overlying improved ground are typically designed using an allowable bearing pressure up to 5,000 psf.

## Shallow Foundation Support

At this time, it is unknown whether there might be small retaining walls or other small structures supported at grade. If small non-pile-supported structures are planned, we recommend that footings be founded on at least 2 feet of structural fill. The zone of structural fill should extend laterally beyond the footing edges a horizontal distance at least equal to the thickness of the fill. An allowable soil bearing pressure of 2,500 psf may be used for the footings, provided that the foundations have a minimum width of 2 feet and bear on a minimum of 2 feet of compacted structural fill. These bearing pressures apply to the sum of all dead plus long-term live loads, excluding the weight of the footing and any overlying backfill. These values may be increased by one-third when wind or seismic loads are considered. Foundation settlement for these support conditions under static loads is estimated to be on the order of ½ to 1 inch. This type of support might result in significant settlement if liquefaction of underlying soils occurs during an earthquake. Foundation settlements if liquefaction occurs could be on the order of 2 to 10 inches, as discussed previously.

We recommend a minimum embedment of 18 inches for shallow foundations for frost depth. As the structural fill will be founded on undocumented existing fill, we strongly recommend that all prepared foundation subgrades be observed by a representative of GeoEngineers to confirm that unsuitable fill (for example, fill containing trash or significant organics/wood debris) is not present.

## Retaining Walls

We recommend that walls for loading docks or other building walls which will serve as retaining walls be designed for lateral pressures based on an equivalent fluid density of 35 pcf. This assumes that the walls will not be restrained against rotation when backfill is placed. The above-recommended lateral soil pressure does not include the effects of surcharges such as floor loads, traffic loads or other surface loading. Surcharge effects should be considered as appropriate.

In settlement-sensitive areas (e.g., beneath on-grade slabs), the upper 2 feet of backfill for subgrade walls should be compacted to at least 95 percent of the MDD determined in accordance with ASTM D-1557. At other locations and below a depth of 2 feet, wall backfill should be compacted to between 90 and 92 percent of ASTM D-1557. Measures should be taken to prevent overcompaction of the backfill behind the wall. This can be achieved by placing the zone of backfill located within 5 feet of the wall in lifts not exceeding 6 inches in loose thickness and compacting this zone with hand-operated equipment such as a vibrating plate compactor.

The recommended equivalent fluid density assumes a free-draining condition behind the wall. This may be achieved by placing an 18- to 24-inch-wide zone of sand and gravel containing less than 5 percent fines against the wall. Weep holes at about 4-foot centers at the base of the wall should be sufficient to drain water from exterior walls. Alternatively, perforated drainpipe could be embedded in the free-draining sand and gravel zone along the base of retaining walls to remove any water which collects in this zone. The drainpipe should be tightlined to an appropriate discharge point.

## Lateral Resistance

The soil resistance available to resist lateral loads is a function of the frictional resistance which can develop on the base of footings and floor slab, and the passive resistance which can develop on the face of below-grade elements of the structure as these elements tend to move into the soil. For footings and floor slabs founded on structural fill placed and compacted in accordance with our recommendations, the allowable frictional resistance may be computed using a coefficient of friction of 0.35 applied to vertical dead-load forces. The allowable passive resistance previously recommended in the Pile Foundations section is appropriate for retaining wall design.

## Earthwork

### Subgrade Preparation

The exposed subgrade should be evaluated after grading is complete and prior to placing base course by proof-rolling with a loaded dump truck. The proof-roll should be observed by a representative from our firm to confirm the subgrade performance. The exposed soil should be firm and unyielding, and without significant groundwater.

If the exposed subgrade is not acceptable based on the proof-roll, we recommend that unsuitable soils be overexcavated to a maximum depth of 2 feet and replaced with imported structural fill.

### Structural Fill

Materials used as fill at the site should meet the requirements below.

- Structural fill placed to support foundations, slabs-on-grade, or driveway, parking and sidewalk areas should meet the requirements of gravel borrow, Washington State Department of Transportation (WSDOT) gravel borrow, WSDOT Standard Specification 9-03.14(1).
- We recommend that structural fill placed for wall or footing drainage systems consist of WSDOT gravel backfill for drains, WSDOT Standard Specification 9-03.12(4).
- Structural fill placed as capillary break material below the floor slab should meet the requirements of WSDOT Standard Specification 9-03.1(4)C, grading No. 57 (1-inch minus crushed rock).

Structural fill must be mechanically compacted to a firm, non-yielding condition. Structural fill must be placed in loose lifts not exceeding 12 inches in thickness. Each lift must be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill must be compacted to the following criteria:

- Structural fill placed to support foundations, slab-on-grade, or driveway, parking and sidewalk areas should be compacted to at least 95 percent of the MDD per ASTM International (ASTM) D 1557.
- Structural fill placed to backfill utility trenches should be compacted to between 90 and 92 percent of the MDD per ASTM D 1557, except for the upper 2 feet that should be compacted to at least 95 percent of MDD.

We recommend that GeoEngineers be present during proof-rolling and/or probing of the exposed subgrade soils in pavement areas, and during placement of structural fill. We will evaluate the adequacy of the subgrade soils and identify areas needing further work, perform in-place moisture-density tests in the fill to verify compliance with the compaction specifications, and advise on any modifications to the procedures that may be appropriate for the prevailing conditions.

#### Reuse of On-site Soils

The on-site soils that will be excavated for construction of the building slab, pile caps, utilities and pavement contain a high percentage of fines; we anticipate that most of the excavated soils will be moisture-sensitive and only be suitable for use in landscaping areas and will not be suitable for reuse as structural fill. On-site soils reused in landscaping areas will likely need amendment to meet landscaping requirements.

If augercast piles are selected as the preferred foundation system, the spoils from construction of the piles will be wet and will need to be disposed of off-site. The spoils will be a mixture of the upper sand, silt, and pile grout and will not be suitable for landscaping areas.

### Temporary Excavations

We anticipate that most excavations required for the project will be relatively shallow, on the order of 4 to 6 feet in depth for the pile caps and utilities. We anticipate that the depth of the excavations required for the pile caps will generally be above the water table. Groundwater may be encountered above this depth if work takes place during or immediately after extended wet weather. We anticipate that the groundwater can be handled during construction by pumping from sumps, as necessary. All collected water should be routed to suitable discharge points.

Excavations deeper than 7 to 8 feet below existing site grades will likely encounter groundwater that will be difficult to handle by sumps alone. A dewatering plan should be developed by the contractor for excavations deeper than about 7 feet.

#### Temporary Cut Slopes

All temporary cut slopes and shoring must comply with the provisions of Title 296 Washington Administrative Code (WAC), Part N, “Excavation, Trenching and Shoring.” The contractor performing the work has the primary responsibility for protection of workers and adjacent improvements.

We recommend temporary cut slope inclinations of 1½H:1V (horizontal to vertical) in the soils encountered at the site. Some caving/sloughing of the cut slopes may occur at this inclination. The inclination may need to be flattened by the contractor if significant caving/sloughing occurs. These cut slope recommendations apply to fully dewatered conditions. For open cuts at the site, we recommend that:

- no traffic, construction equipment, stockpiles or building supplies be allowed at the top of the cut slopes within a distance of at least 5 feet from the top of the cut;
- exposed soil along the slope be protected from surface erosion using waterproof tarps, plastic sheeting or flashcoating with shotcrete;
- construction activities be scheduled so that the length of time the temporary cut is left open is reduced to the extent practicable;
- erosion control measures be implemented as appropriate such that runoff from the site is reduced to the extent practicable;
- surface water be diverted away from the excavation; and
- the general condition of the slopes should be observed periodically by GeoEngineers to confirm adequate stability.

Because the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. The contractor should take all necessary steps to ensure the safety of the workers near slopes.

#### Temporary Shoring

Because of the diversity of available shoring systems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. The following paragraphs present recommendations for the type of shoring systems and design parameters that we conclude are appropriate for the subsurface conditions at the site.

The soils within the project area can be retained using conventional trench shoring systems such as trench boxes, sheet piles, a braced system, or a slide rail system. The design of temporary shoring should allow for lateral pressures exerted by the adjacent soil, surcharge loads from traffic, construction equipment and temporary stockpiles adjacent to the excavation, etc.

The lateral soil pressures acting on temporary shoring will depend on the nature and density of the soil behind the wall, the inclination of the ground surface behind the wall, and the groundwater level. For walls that are free to yield at the top at least one thousandth of the height of the wall (i.e., wall height times 0.001), soil pressures will be less than if movement is restrained. Lateral load resistance can be mobilized through the use of braces, tiebacks, anchor blocks and passive pressures on members that extend below the bottom of the excavation. Temporary shoring used to support trench excavations typically uses internal bracing such as hydraulic shoring or trench boxes.

We recommend that yielding walls retaining the existing soils be designed using an equivalent fluid density of 40 pcf, for horizontal ground surfaces. For non-yielding (i.e., braced) systems, we recommend that the shoring be designed for a uniform lateral pressure of 26H in psf, where H is the depth of the planned excavation in feet below a level ground surface. These values assume that the ground behind the shoring has been dewatered such that the ground water table is at least 2 feet below the base of the excavation.

If the dewatering system is not designed to lower the groundwater level behind the shoring walls (e.g. sheet pile walls with dewatering system inside the shored excavation), hydrostatic pressures must be included in the shoring design. For this condition, temporary shoring should be designed using a lateral pressure equal to an equivalent fluid density of 85 pcf, for horizontal ground conditions adjacent to the excavation.

The above lateral soil pressures do not include traffic, structure or construction surcharges that should be added separately, if appropriate.

The soil pressure available to resist lateral loads against shoring is a function of the passive resistance that can develop on the face of below-grade elements of the shoring as those elements move horizontally into the soil. The allowable passive resistance on the face of embedded shoring elements may be computed using an equivalent fluid density of 125 pcf. This passive equivalent fluid density value is for soil below the water table and includes a factor of safety of about 1.5.

### Weather Considerations

The on-site soils generally contain a sufficiently high percentage of fines (silt and clay) and are therefore moisture-sensitive. When the moisture content of these soils is more than a few percent above the optimum moisture content, these soils become muddy and unstable, operation of equipment on these soils will be difficult, and it will be difficult or impossible to meet the required compaction criteria. Additionally, disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather. It will be preferable to schedule site preparation and earthwork activities during extended periods of dry weather when the soils will: (1) be less susceptible to disturbance; (2) provide better support for construction equipment; and (3) be more likely to meet the required compaction criteria.

The wet weather season in western Washington generally begins in October and continues through May; however, periods of wet weather may occur during any month of the year. The optimum earthwork period for these types of soils is typically June through September. If wet weather earthwork is unavoidable, we recommend the following:

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded such that areas of ponded water do not develop. The contractor should take measures to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site with appropriate best management practices (BMPs) to control sedimentation.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.

### Preliminary Infiltration Considerations

We understand infiltration facilities are being considered in the proposed parking areas. Preliminary infiltration rates were estimated based on grain size analyses using the guidelines in the Stormwater Management Manual for Western Washington (SMMWW) adopted by City of Issaquah. The preliminary rates should be confirmed by pilot infiltration testing when the facility location and depth is determined. Based on our experience, the rates calculated by the grain size method are typically higher than in-situ measurements.

**TABLE 6. PRELIMINARY INFILTRATION RATES**

| Exploration | Soil Type | Depth (feet) | Short-Term Infiltration Rate (in/hr) uncorrected | Correction Factor CF[1] | Estimated Design (Long-term) Infiltration Rate (in/hr) |
|---|---|---|---|---|---|
| TP-2 | Gravel with Silt and Sand | 5 | 19.7 | 0.119 | 2.34 |
| TP-4 | Silty Sand | 1 | 9.7 | 0.119 | 1.15 |
| TP-4 | Sand with Silt | 5 | 52.5 | 0.119 | 6.24 |
| TP-5 | Gravel with Silt and Sand | 2 | 31.0 | 0.119 | 3.69 |
| TP-8 | Silty Sand | 2 | 12.0 | 0.119 | 1.43 |
| MW-2 | Sand with Silt | 5 | 46.4 | 0.119 | 5.51 |
| MW-3 | Silty Sand | 5 | 5.7 | 0.119 | 0.68 |

Notes:

[1] Total Correction Factor based on $CF_v$ = 0.33, $Cf_t$ = 0.4 and long-term conductivity loss factor = 0.9

### Utilities

Trench excavation, pipe bedding, and trench backfilling should be completed using the general procedures described in the WSDOT Standard Specifications or other suitable procedures specified by the project civil engineer. Utility pipes should be bedded with bedding material as specified by the project civil engineer. We recommend a minimum 6-inch-thick layer, or one-fourth of the pipe diameter, whichever is greater, of pipe bedding material be placed below, above, and around the perimeter of the pipe. This bedding material should be lightly tamped into place. Backfill placed above the bedding material shall consist of structural fill quality material as discussed above.

Utility trench backfill should be placed in lifts of 12 inches or less (loose thickness) such that adequate compaction can be achieved throughout the entire lift. Each lift must be compacted prior to placing the subsequent lift. Prior to compaction, the backfill should be moisture conditioned to near optimum moisture content, if necessary. The backfill should be compacted in accordance with the criteria discussed above.

#### Dewatering

We recommend that the ground water level be maintained 1 to 2 feet below the bottom of excavations during construction, or that level necessary to stabilize the shoring and provide a firm subgrade. Quarry spalls and pea gravel can be used as bedding for utilities that extend below the groundwater level. The groundwater level will depend upon the dewatering method, the size of the excavation and other factors. We do not anticipate that a significant dewatering effort will be required during construction of shallow utilities. However, vaults or tanks extending below the groundwater level may be required, in which case more extensive dewatering will be necessary (well points or deep wells). Any seepage that enters the shallow utility excavations can likely be handled by the use of sumps and pumps.

#### Buoyancy

The effects of buoyancy should be considered in design of the utilities and vaults extending deeper than 8 feet bgs. Buoyancy effects can be resisted by the dead weight of the structure, friction along the sides of the structure, and the weight of zones of soil which are located above the slab floor which protrude beyond the permanent walls. Frictional resistance can be computed using a coefficient of friction of 0.4 applied to the lateral soil pressures. This coefficient of friction value includes a factor of safety of about 1.5. We recommend that lateral soil pressure for uplift resistance be computed using an equivalent fluid density of 20 pcf considering groundwater is present. Backfill above the slab floor may be assumed to have a submerged unit weight of 57 pcf.

### Pavement Recommendations

#### Subgrade Preparation

Pavement subgrade areas should be stripped and proofrolled, or probed to evaluate the existing subgrade surface prior to placing new fill for pavement support or the new pavement section. Where the existing soils are loose or wet and cannot be compacted, it will be necessary to excavate and replace these soils. The required excavation thickness will depend on the moisture content of the subgrade soils at the time of construction and should be evaluated at that time. To avoid the cost of additional overexcavation, the pavement subgrade preparation should occur during the dry season as practical.

#### Design Section

Based on our experience with similar developments, we recommend the following minimum pavement design sections. The heavier section should be utilized throughout the site if automobile parking areas cannot be strictly designated.

**TABLE 7. RECOMMENDED DESIGN PAVEMENT SECTIONS**

| Pavement Area | HMA CL. ½ PG 64-22[1] (inches) | Crushed Surfacing Base Course with less than 5 percent fines content[2] (inches) |
|---|---|---|
| Automobile Parking | 2 | 6 |
| Entrance Drive and Heavier Truck Traffic | 3 | 6 |

Notes:

[1] Hot mix asphalt (HMA) Class ½-inch, PG 64-22 per WSDOT Standard Specification 5-04 and 9-03. Minimum 2-inch thickness recommended.

[2] Crushed Surfacing per WSDOT Standard Specification 9-03.9(3) compacted to 95 percent of the MDD determined using ASTM D-1557, to contain less than 5 percent fines content and to be placed on subgrade compacted to 95 percent of MDD.

### Drainage Considerations

We recommend that pavement surfaces be sloped so that surface drainage flows away from the building, and all roof drainage be collected in tight lines for diversion into the storm drain system. A perimeter footing drain is recommended to intercept surface water runoff that may be perched on the surficial silty sand soils. All areas should be graded to avoid concentration of runoff onto fill or cut slopes or other erosion-sensitive areas.

### Recommended Additional Geotechnical Services

Throughout this report, recommendations are provided where we consider additional geotechnical services to be appropriate. These additional services are summarized below:

- Additional insitu testing (pilot infiltration testing) should be completed at site specific infiltration facilities to confirm infiltration rates.
- If augercast piles are selected as the foundation system, we recommend additional explorations be completed to refine the required pile embedment depths across the footprint.
- GeoEngineers should be retained to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.
- During construction, GeoEngineers should observe and document installation of deep foundations or ground improvement, evaluate the suitability of the pavement and slab subgrades, observe and test structural backfill and provide a summary letter of our construction observation services. The purposes of GeoEngineers construction phase services are to confirm that the subsurface conditions are consistent with those observed in the explorations and other reasons described in Appendix C, Report Limitations and Guidelines for Use.

## LIMITATIONS

We have prepared this report for the exclusive use of Strotkamp Associates. and members of the design team for the Evergreen Ford Lincoln property in Issaquah, Washington. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix C, Report Limitations and Guidelines for Use for additional information pertaining to use of this report.

## REFERENCES

American Association of State Highway and Transportation Officials, "LRFD Bridge Design Specifications."

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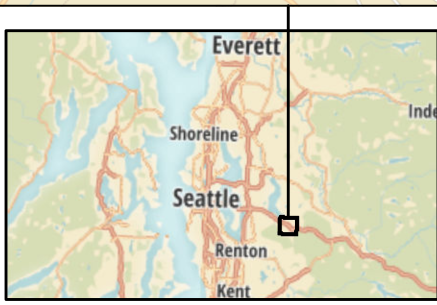
Youd, T. L. and Idriss, I. M. 2001. "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 4, April 2001, pp. 298-313.

FIGURES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Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

**Vicinity Map**

Evergreen Ford Lincoln
Issaquah, Washington

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**Figure 1**

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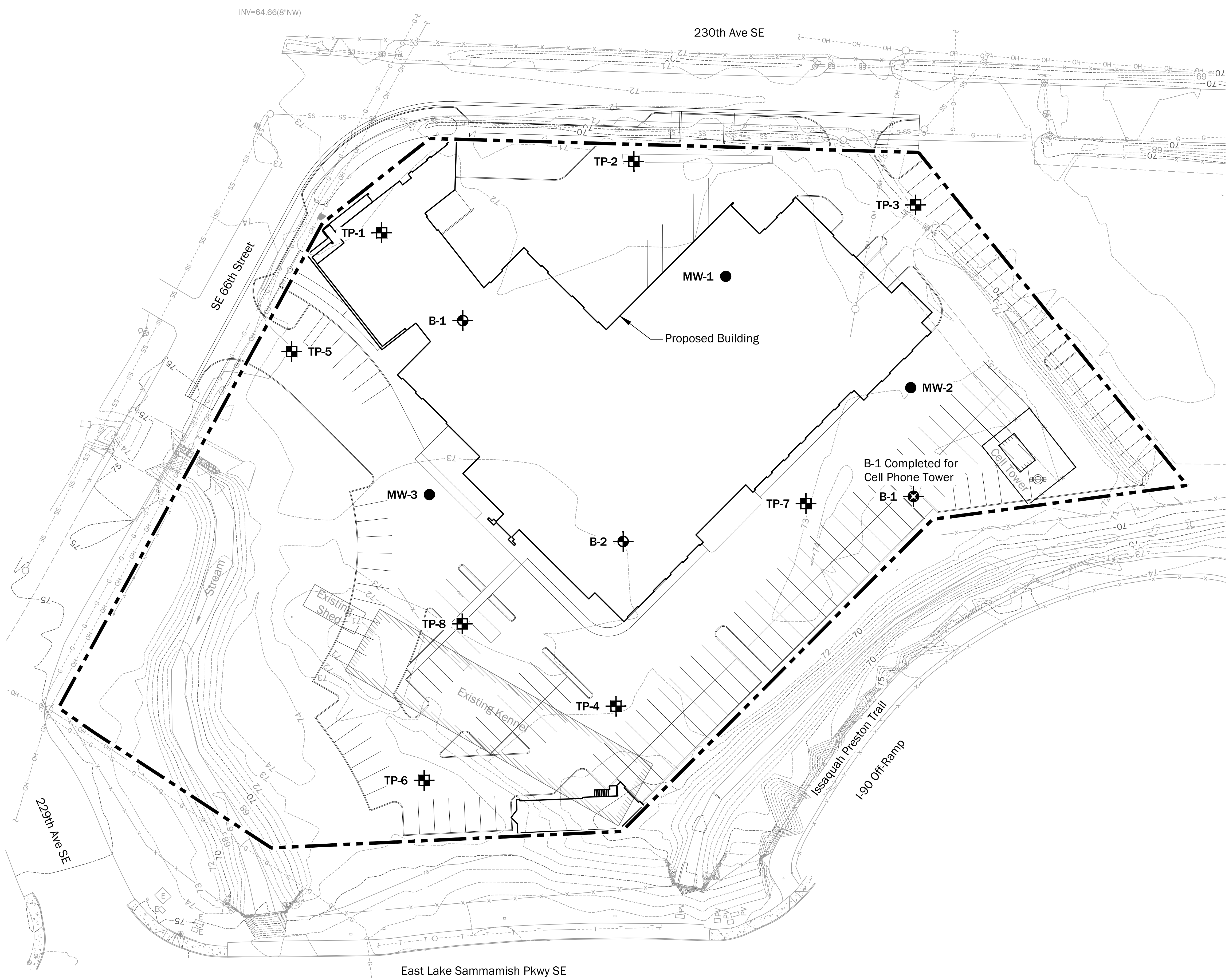


## Legend

- Property Boundary
- GEI-X — Boring by GeoEngineers, 2018
- TP-X — Test Pit by GeoEngineers, 2018
- MW-X — Monitoring Well by GeoEngineers, 2018
- B-1 — Boring by Others, 1997

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Background received from SCJ Alliance on 12/3/2018.

Projection: WA State Plane, North Zone, NAD83, US Foot

60 0 60
Feet

**Site Plan**

Evergreen Ford Lincoln
Issaquah, Washington

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**Figure 2**

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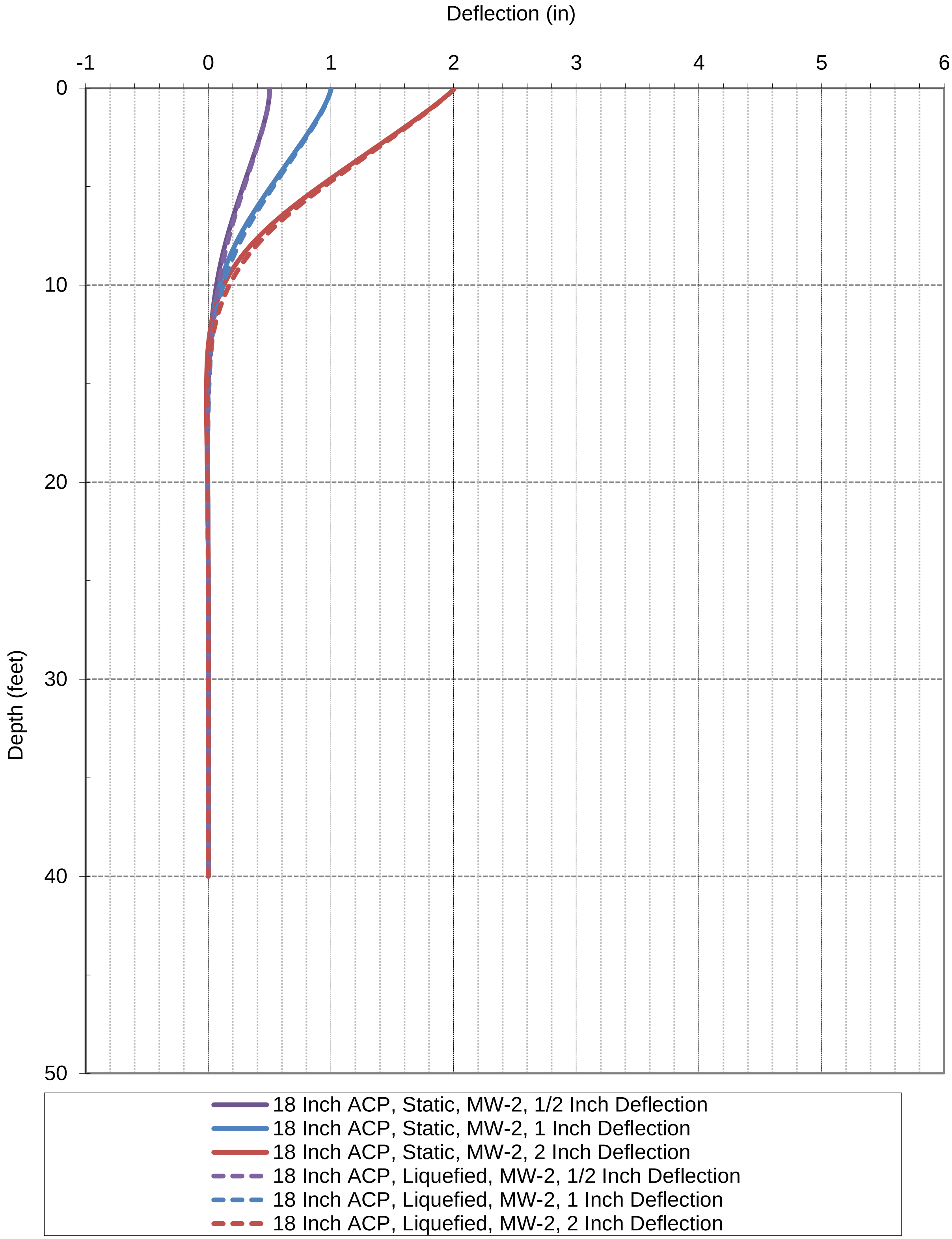


**18 Inch Augercast Pile**
**Deflection vs. Depth MW-2**

Evergreen Ford Lincoln
Issaquah, Washington

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**Figure 3**

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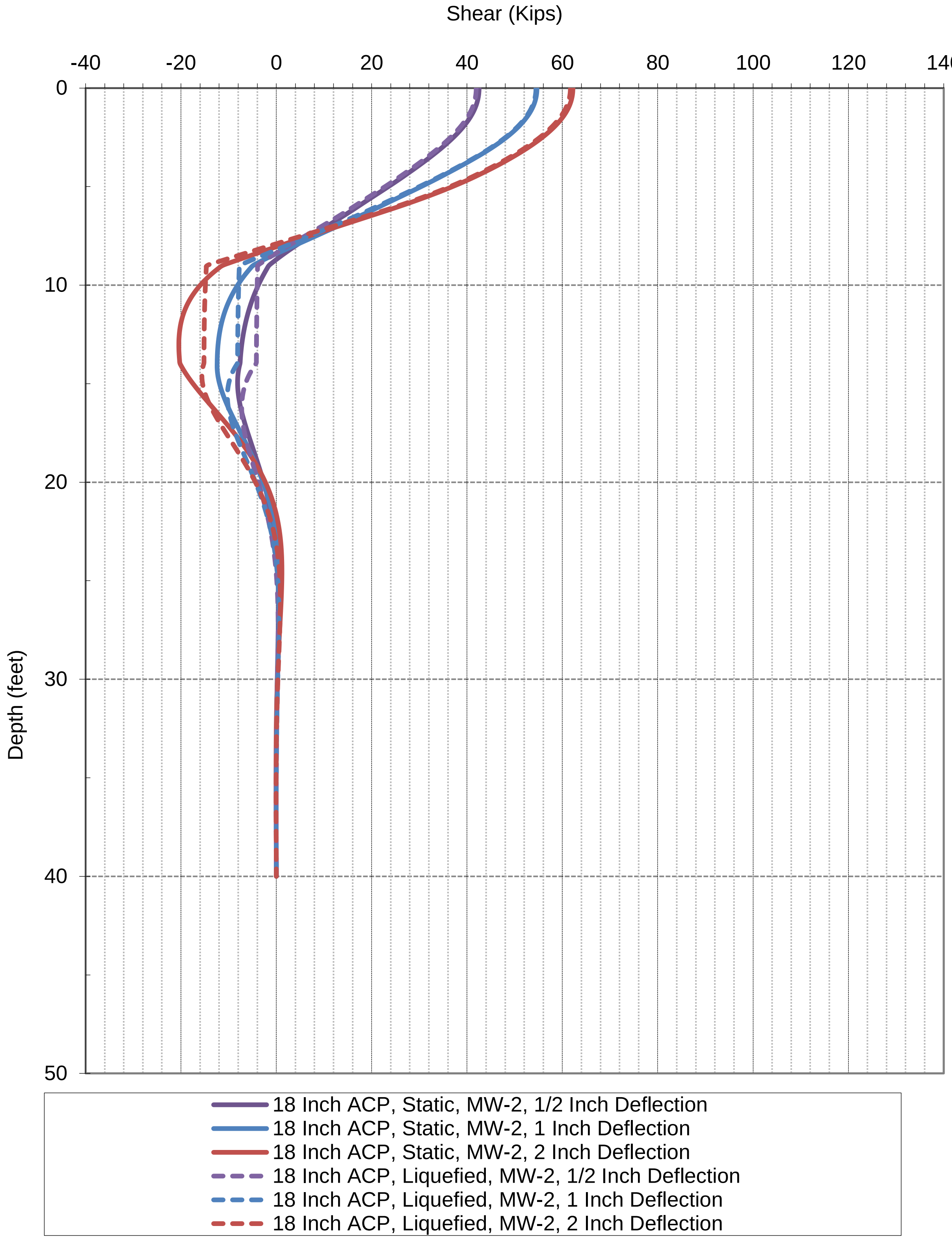


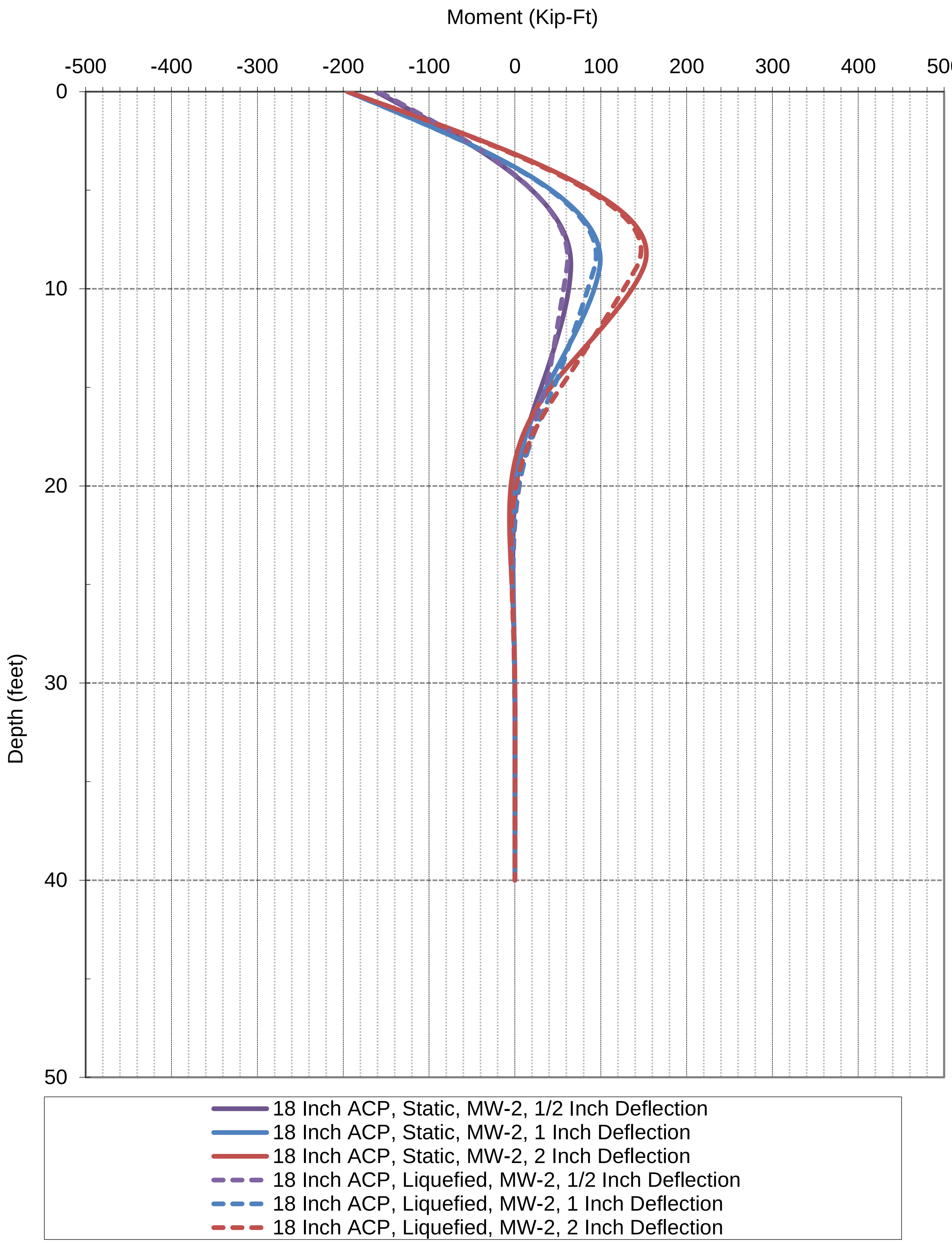

**18 Inch Augercast Pile**
**Shear vs. Depth MW-2**

Evergreen Ford Lincoln
Issaquah, Washington

GeoEngineers

**Figure 4**

Sharepoint 23589-001-00\Technical Analysis DTM: 1/10/2019

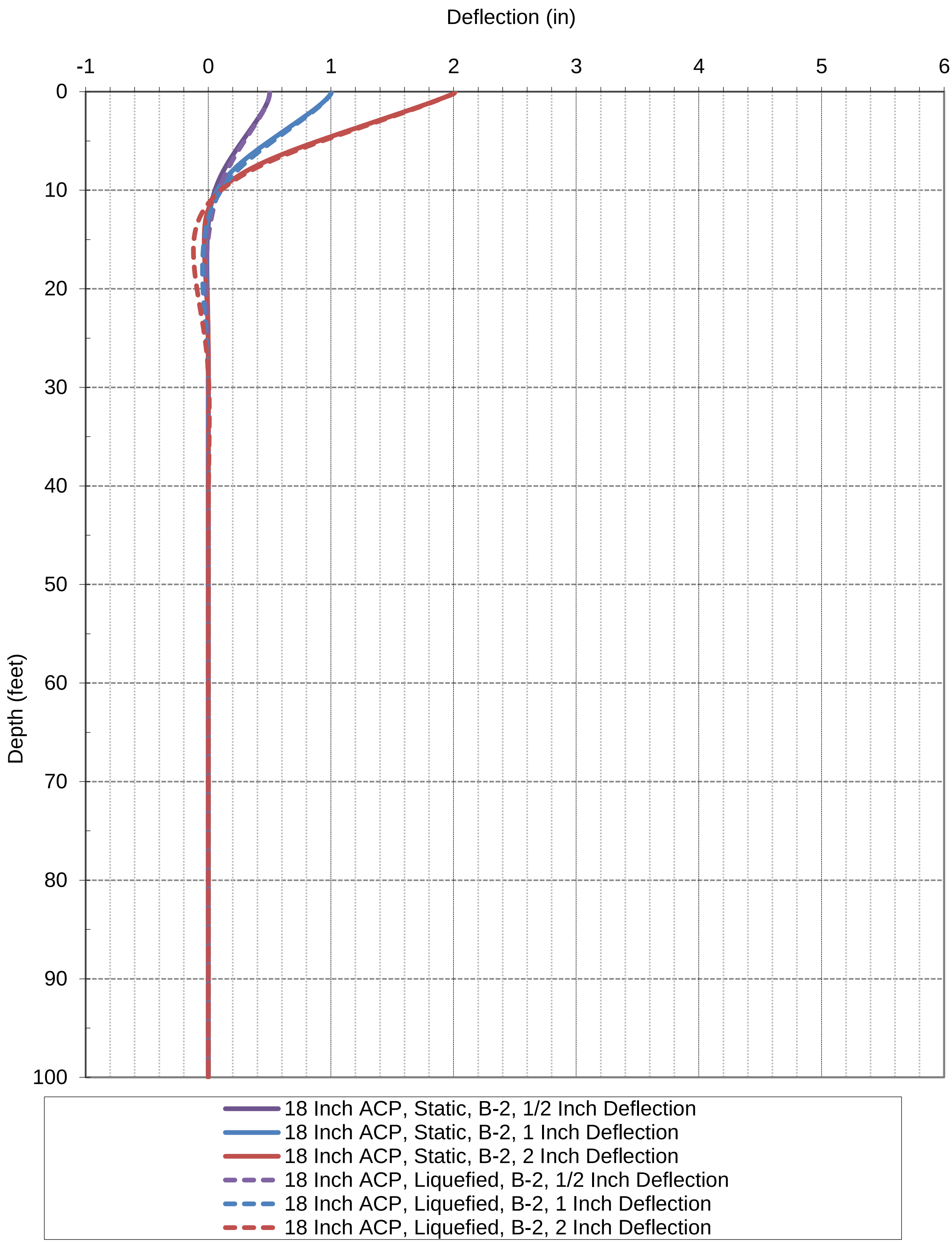



**18 Inch Augercast Pile**
**Moment vs. Depth MW-2**

Evergreen Ford Lincoln
Issaquah, Washington

GeoEngineers

**Figure 5**




**18 Inch Augercast Pile**
**Deflection vs. Depth B-2**

**Evergreen Ford Lincoln**
**Issaquah, Washington**

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**Figure 6**

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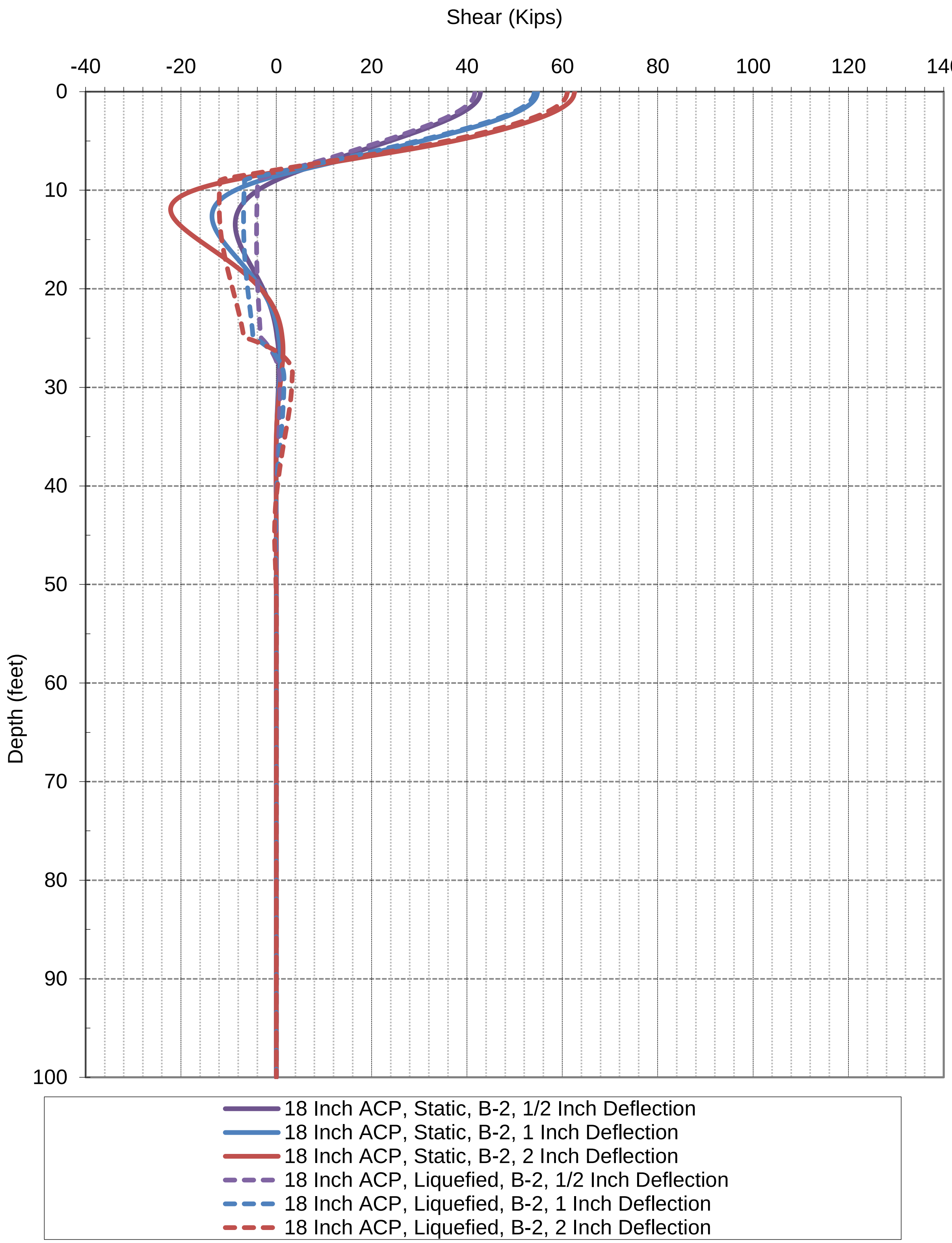


**18 Inch Augercast Pile**
**Shear vs. Depth B-2**

Evergreen Ford Lincoln
Issaquah, Washington

GeoEngineers

**Figure 7**

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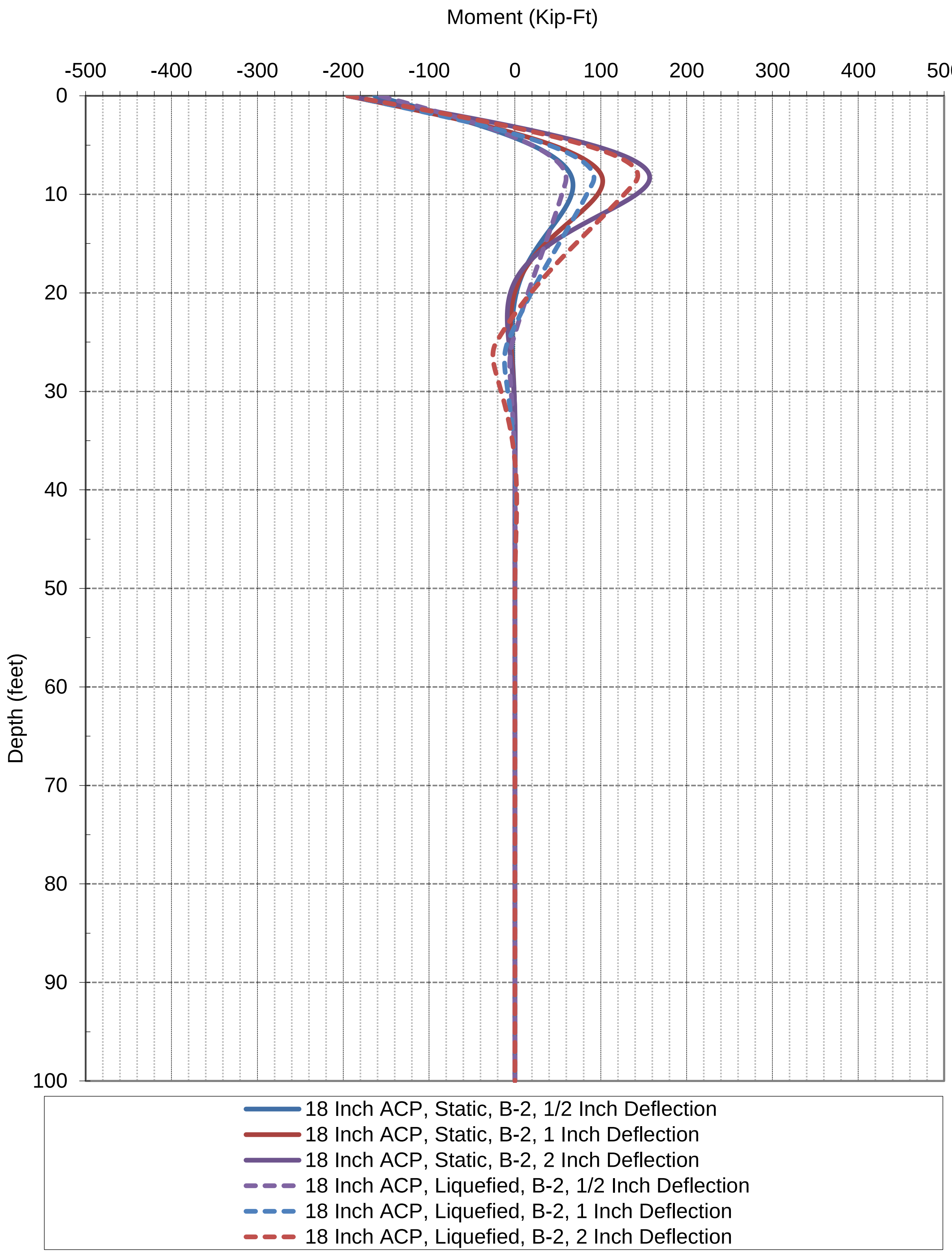


**18 Inch Augercast Pile**
**Moment vs. Depth B-2**

**Evergreen Ford Lincoln**
**Issaquah, Washington**

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**Figure 8**

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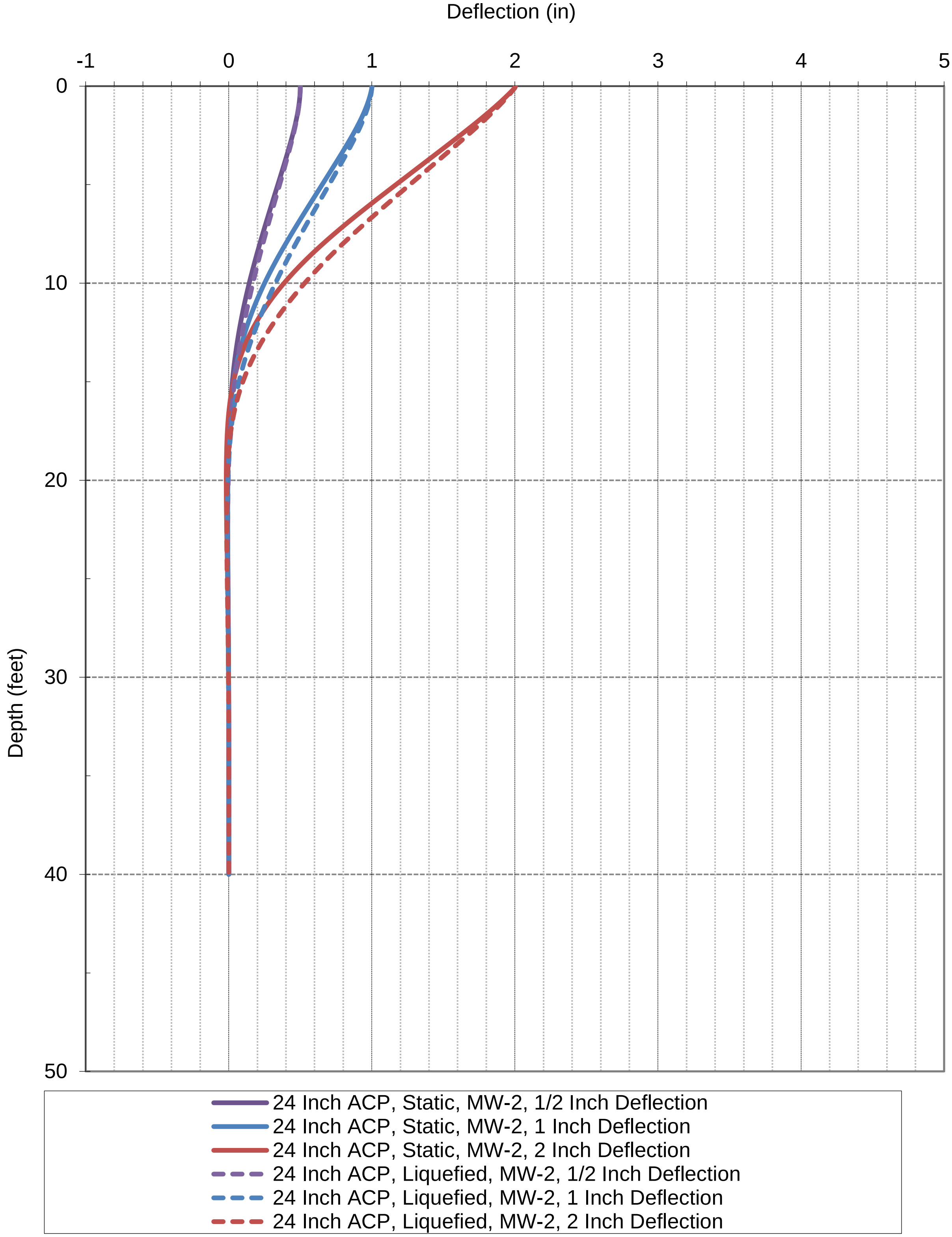


**24 Inch Augercast Pile**
**Deflection vs. Depth MW-2**

Evergreen Ford Lincoln
Issaquah, Washington

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**Figure 9**

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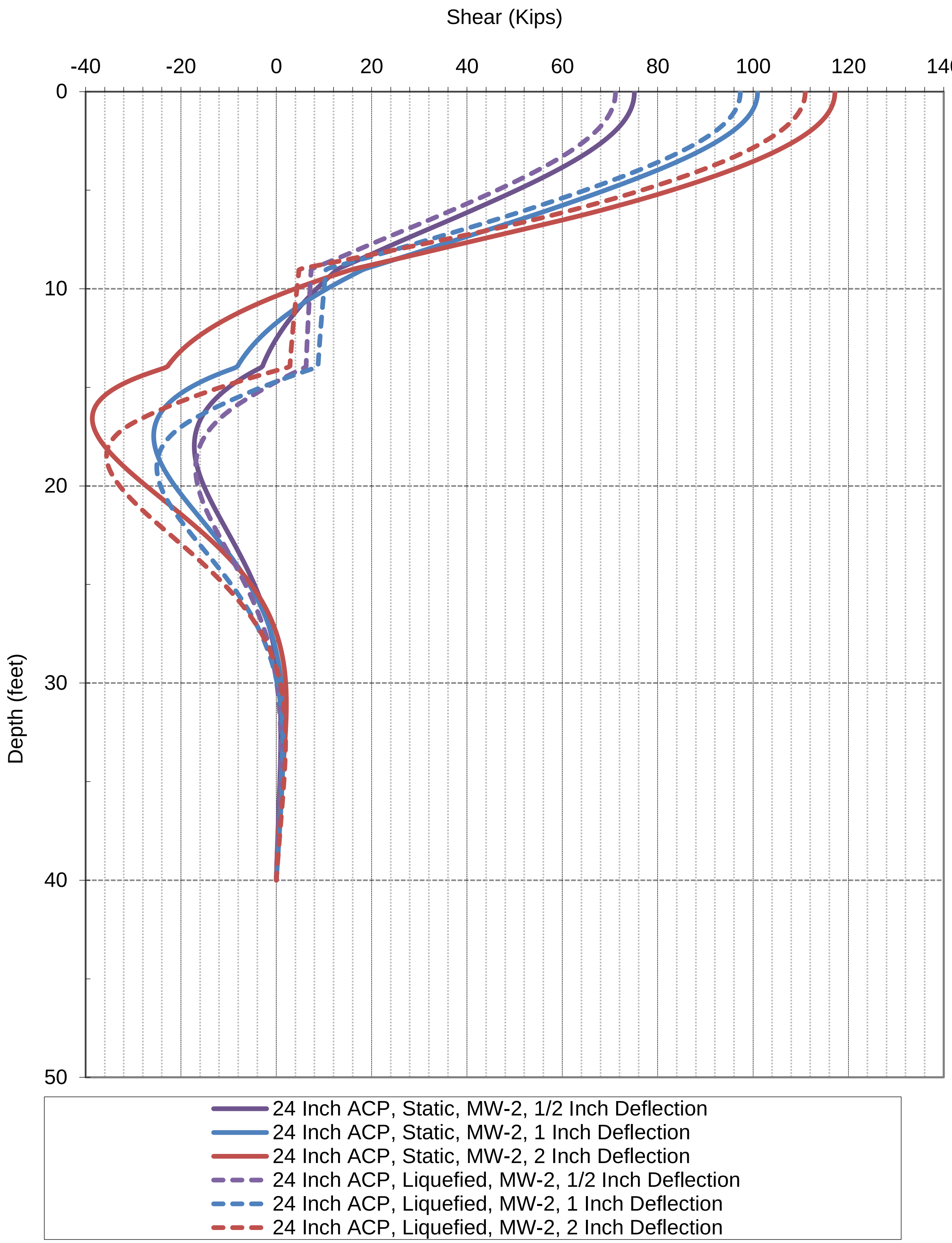


**24 Inch Augercast Pile**
**Shear vs. Depth MW-2**

Evergreen Ford Lincoln
Issaquah, Washington

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**Figure 10**

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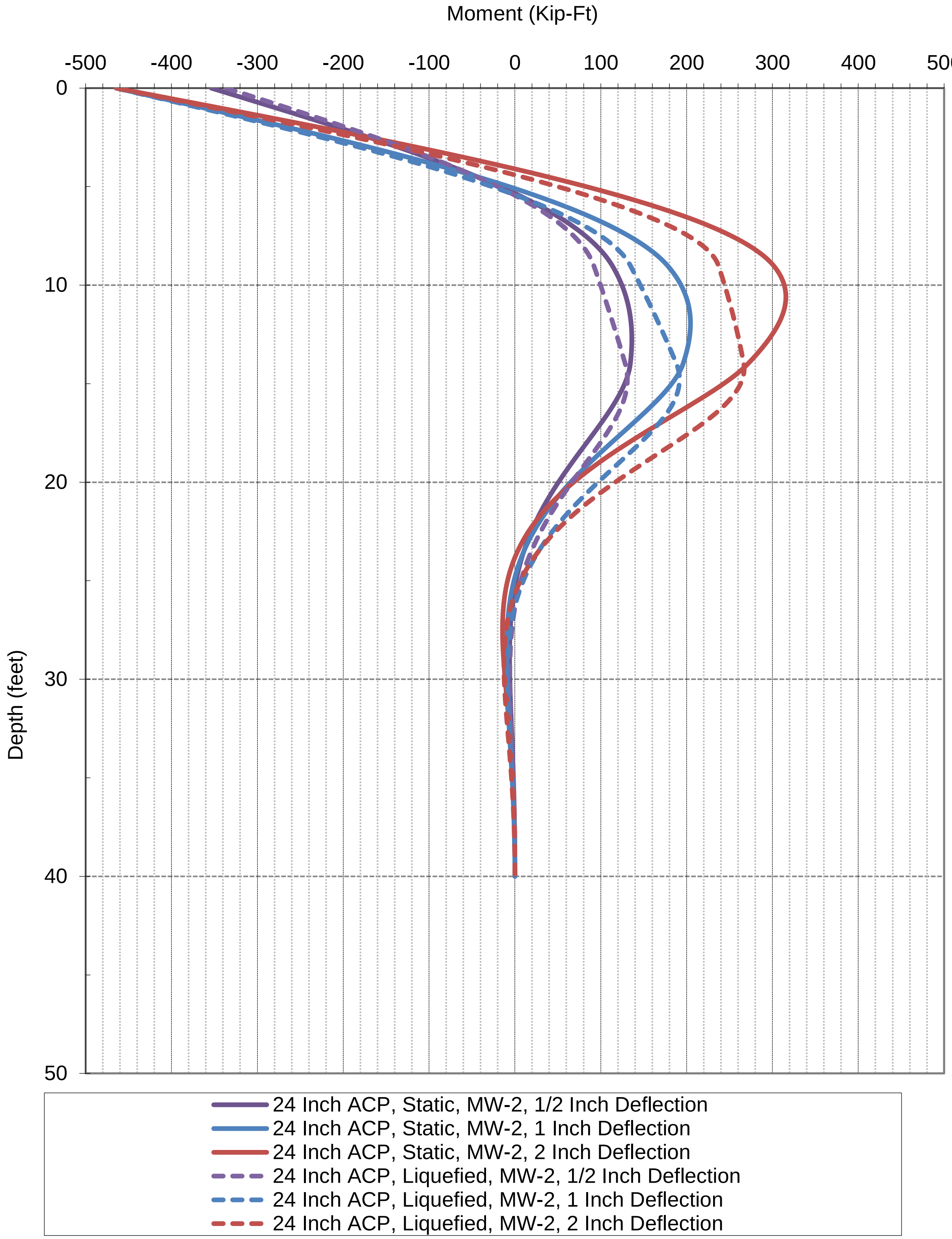


**24 Inch Augercast Pile**
**Moment vs. Depth MW-2**

**Evergreen Ford Lincoln**
**Issaquah, Washington**

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**Figure 11**

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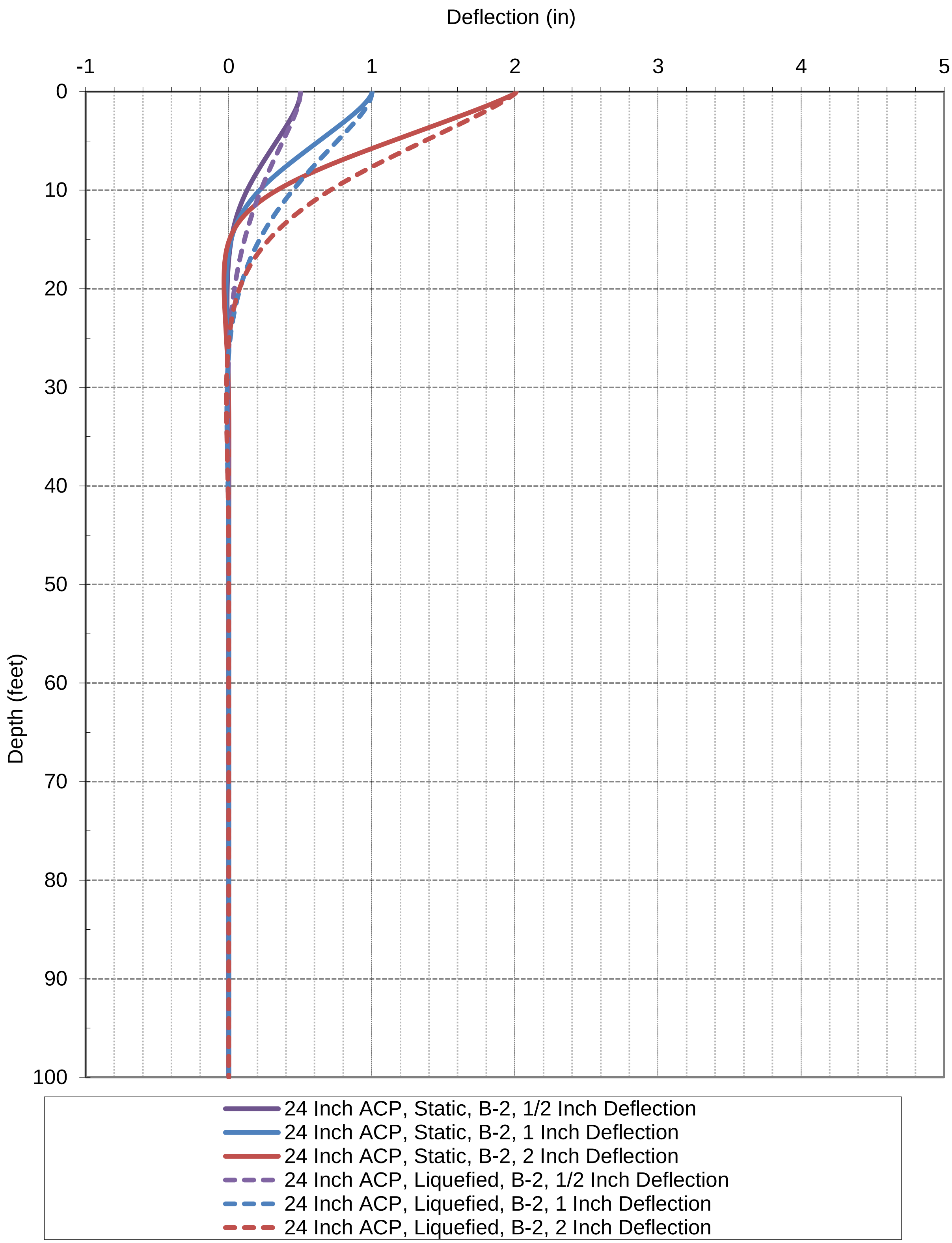


**24 Inch Augercast Pile**
**Deflection vs. Depth B-2**

**Evergreen Ford Lincoln**
**Issaquah, Washington**

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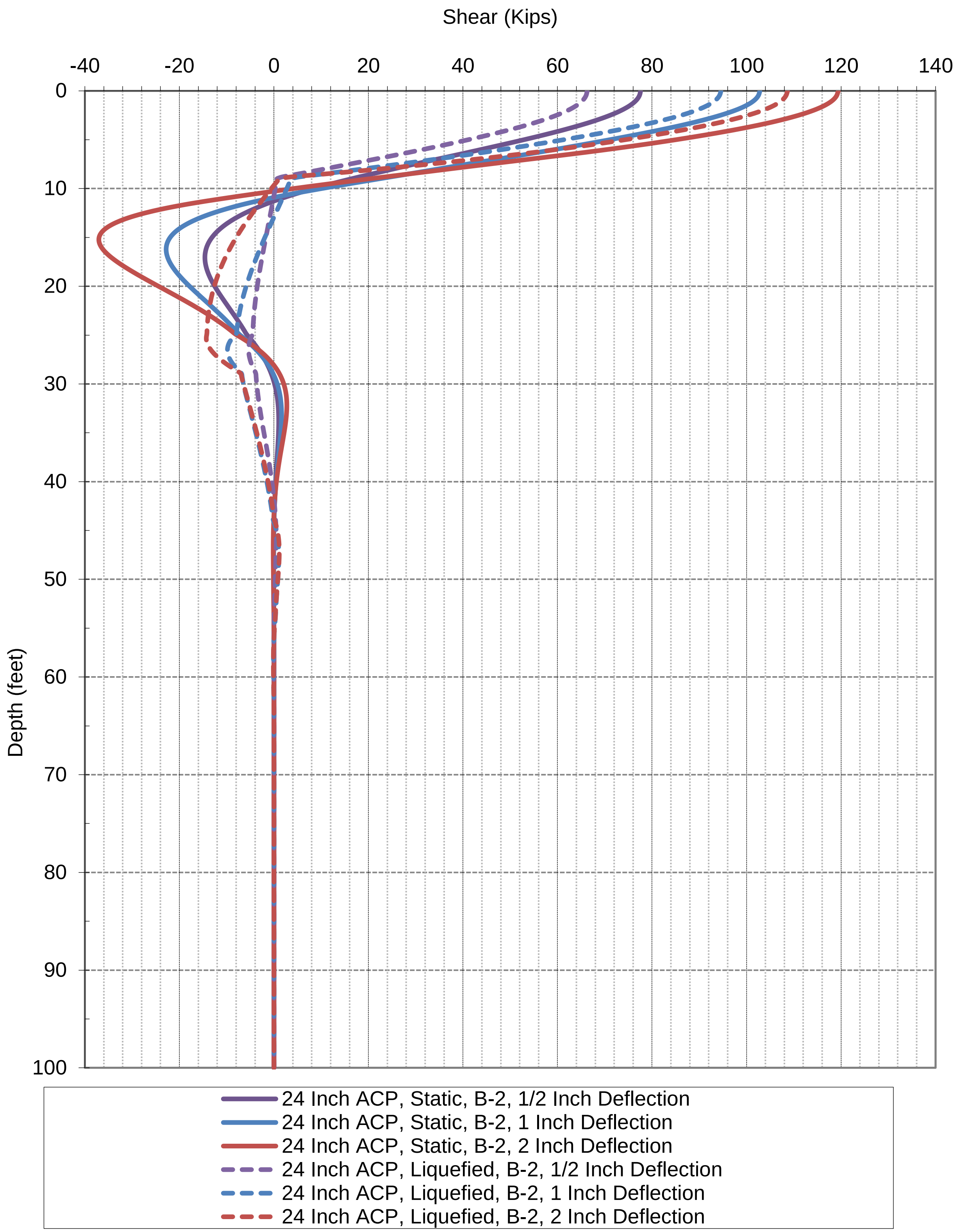
**Figure 12**

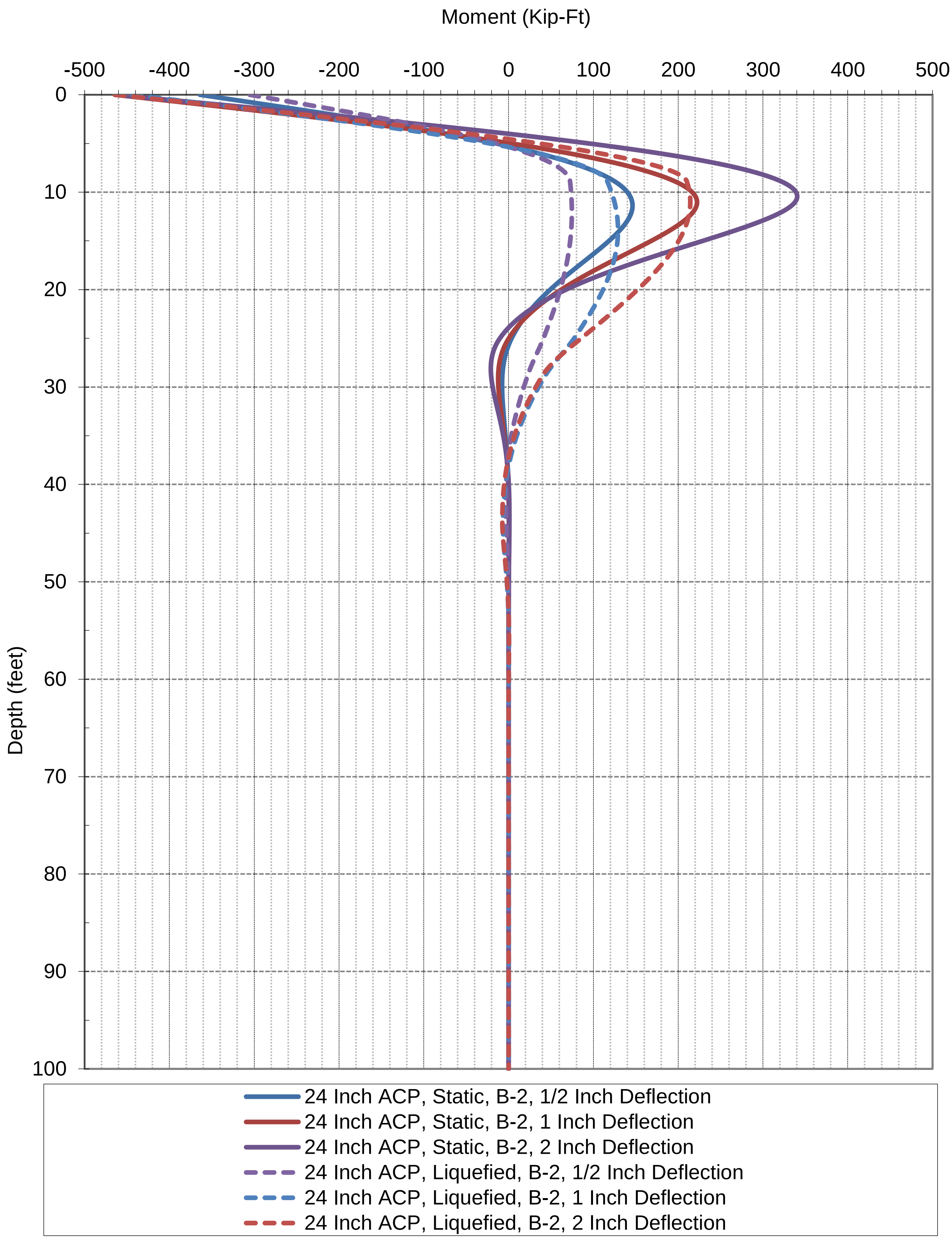



**24 Inch Augercast Pile**
**Shear vs. Depth B-2**

**Evergreen Ford Lincoln**
**Issaquah, Washington**

GeoEngineers

**Figure 13**




**24 Inch Augercast Pile**
**Moment vs. Depth B-2**

**Evergreen Ford Lincoln**
**Issaquah, Washington**

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**Figure 14**

APPENDICES

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# APPENDIX A
## Field Explorations and Laboratory Testing

## APPENDIX A
## FIELD EXPLORATIONS AND LABORATORY TESTING

### Field Explorations

Subsurface conditions at the site were explored on October 31 through November 2, 2018 by drilling five borings/monitoring wells (MW-1 through MW-3 and B-1 and B-2) at the approximate locations shown on Figure 2, and by completing eight test pits across the site. The approximate exploration locations were established in the field by measuring distances from existing site features and using a handheld GPS. The explorations were completed to depths between 5 and 50 feet using track-mounted equipment owned and operated by Saber and Advanced Drill Technologies.

#### Borings/Monitoring Wells

Disturbed soils samples were obtained during drilling using standard penetration test (SPT) methodology with the standard split-spoon sampler in the borings. The samples were placed in plastic bags to maintain the moisture content and transported back to our laboratory for analysis and testing.

The borings were continuously monitored by a geologist from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions and prepared a detailed log of each exploration. Soils encountered were classified visually in general accordance with ASTM D2488-09a the classification system described in Figure A-1. An explanation of our boring log symbols is also shown on Figure A-1.

The logs of the borings are presented in Figures A-2 through A-6. The exploration logs are based on our interpretation of the field and laboratory data and indicate the various types of soils encountered. The logs also indicate the depths at which these soils or their characteristics change, although the change might actually be gradual. If the change occurred between samples in the boring, it was interpreted.

#### Test Pits

Eight test pit explorations were completed to observe shallow surface conditions such as thickness of fill, groundwater seepage, soil density, and existence of compressible soils. Soil description, probe depths, groundwater observations, caving conditions, and field measured shear strength measurements are recorded on test pit logs. The logs of the test pits are presented in Figures A-7 through A-14.

### Laboratory Testing

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate index properties of the soil samples. Representative samples were selected for laboratory testing consisting of the determination of the moisture content, sieve analyses, and percent fines. The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

#### Moisture Content Testing

Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs at the depths at which the samples were obtained.

### Sieve Analyses

Sieve analyses were performed on selected samples in general accordance with ASTM D 422 to determine the sample grain size distribution. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted, classified in general accordance with the Unified Soil Classification System (USCS), and are presented in Figures A-15 and A-16.

### Percent Fines Test

Percent fines (particles passing the No. 200 sieve) were completed on soil samples using ASTM D 1140. The wet sieve method was used to determine the percentage of soil particles larger than the U.S. No. 200 sieve opening. The results of the percent fines tests are presented on the boring logs at the depths at which the samples were obtained.

## SOIL CLASSIFICATION CHART

| MAJOR DIVISIONS | | | SYMBOLS GRAPH | SYMBOLS LETTER | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|---|
| COARSE GRAINED SOILS<br>MORE THAN 50% RETAINED ON NO. 200 SIEVE | GRAVEL AND GRAVELLY SOILS<br>MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE | CLEAN GRAVELS<br>(LITTLE OR NO FINES) | | GW | WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES |
| | | | | GP | POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES |
| | | GRAVELS WITH FINES<br>(APPRECIABLE AMOUNT OF FINES) | | GM | SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES |
| | | | | GC | CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES |
| | SAND AND SANDY SOILS<br>MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE | CLEAN SANDS<br>(LITTLE OR NO FINES) | | SW | WELL-GRADED SANDS, GRAVELLY SANDS |
| | | | | SP | POORLY-GRADED SANDS, GRAVELLY SAND |
| | | SANDS WITH FINES<br>(APPRECIABLE AMOUNT OF FINES) | | SM | SILTY SANDS, SAND - SILT MIXTURES |
| | | | | SC | CLAYEY SANDS, SAND - CLAY MIXTURES |
| FINE GRAINED SOILS<br>MORE THAN 50% PASSING NO. 200 SIEVE | SILTS AND CLAYS | LIQUID LIMIT LESS THAN 50 | | ML | INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY |
| | | | | CL | INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS |
| | | | | OL | ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY |
| | SILTS AND CLAYS | LIQUID LIMIT GREATER THAN 50 | | MH | INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS |
| | | | | CH | INORGANIC CLAYS OF HIGH PLASTICITY |
| | | | | OH | ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY |
| HIGHLY ORGANIC SOILS | | | | PT | PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS |

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

### Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

## ADDITIONAL MATERIAL SYMBOLS

| SYMBOLS GRAPH | SYMBOLS LETTER | TYPICAL DESCRIPTIONS |
|---|---|---|
| | AC | Asphalt Concrete |
| | CC | Cement Concrete |
| | CR | Crushed Rock/ Quarry Spalls |
| | SOD | Sod/Forest Duff |
| | TS | Topsoil |

### Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

### Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

### Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

### Laboratory / Field Tests

| | |
|---|---|
| %F | Percent fines |
| %G | Percent gravel |
| AL | Atterberg limits |
| CA | Chemical analysis |
| CP | Laboratory compaction test |
| CS | Consolidation test |
| DD | Dry density |
| DS | Direct shear |
| HA | Hydrometer analysis |
| MC | Moisture content |
| MD | Moisture density |
| Mohs | Mohs hardness scale |
| OC | Organic content |
| PM | Permeability or hydraulic conductivity |
| PI | Plasticity index |
| PP | Pocket penetrometer |
| SA | Sieve analysis |
| TX | Triaxial compression |
| UC | Unconfined compression |
| VS | Vane shear |

### Sheen Classification

| | |
|---|---|
| NS | No Visible Sheen |
| SS | Slight Sheen |
| MS | Moderate Sheen |
| HS | Heavy Sheen |

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

# Key to Exploration Logs

GEOENGINEERS

Figure A-1

| Drilled | Start: 11/1/2018 | End: 11/1/2018 | Total Depth (ft) | 51.5 | Logged By: WCW<br>Checked By: MSH | Driller | Advanced Drill Technologies | Drilling Method | Hollow-stem Auger |
|---|---|---|---|---|---|---|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| Surface Elevation (ft)<br>Vertical Datum | 72<br>NAVD88 | Hammer Data | Autohammer<br>140 (lbs) / 30 (in) Drop | Drilling Equipment | Diedrich D50 Turbo |
| Latitude<br>Longitude | 47.54232<br>-122.03412 | System Datum | WA State Plane North<br>NAD83 (feet) | See "Remarks" section for groundwater observed | |

Notes:

| Elevation (feet) | Depth (feet) | FIELD DATA: Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | SM | Brown silty fine sand, oxidation staining (loose, moist) (fill) | | | |
| 70 | | | | | | | | | | | | |
| | | X | 12 | 5 | | 1 | | | | | | |
| | | | | | | | | | Becomes tan | | | |
| | 5 | X | 10 | 7 | | 2a<br>2b<br>MC | | | | 28 | | |
| 65 | | | | | | | | | | | | |
| | | | | | | | | GM | Brown silty fine to coarse gravel with sand (medium dense, wet) | | | Groundwater observed at approximately 8½ feet at time of drilling |
| | 10 | X | 10 | 13 | | 3a<br>3b | | | | | | |
| 60 | | | | | | | | | Grades without gravel | | | |
| | | | | | | | | SP-SM | Gray fine to coarse sand with silt and gravel (medium dense, wet) | | | |
| | 15 | X | 8 | 13 | | 4<br>%F | | | | 12 | 5 | |
| 55 | | | | | | | | | | | | |
| | | | | | | | | SM | Gray-blue silty fine sand with occasional gravel (medium dense, wet) | | | |
| | 20 | X | 18 | 13 | | 5a<br>5b | | | | | | Driller added mud |
| 50 | | | | | | | | | | | | |
| | | | | | | | | SM | Blue-gray silty fine to coarse sand with gravel (medium dense to dense, wet) | | | |
| | 25 | X | 4 | 30 | | 6 | | | | | | |
| 45 | | | | | | | | | | | | |
| | | | | | | | | SM | Brown silty fine sand with 2-inch sand lens (loose, wet) | | | |
| | 30 | X | 18 | 8 | | 7 | | | | | | |
| 40 | | | | | | | | SM | Brown silty fine to coarse sand with occasional gravel (medium dense, wet) | | | |
| | 35 | | | | | | | | | | | |

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.



## Log of Boring B-1

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

| Elevation (feet) | Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name Testing | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 35 | | 12 | 14 | | 8 | | | | | | Silt content decreased with depth as sand content increased, gravel at top of sample |
| 35 | | | | | | | | GP-GM | Brown fine to coarse gravel with silt, sand and occasional cobbles (dense to very dense, wet) | | | |
| | 40 | | 16 | 49 | | 9 | | | | | | |
| 30 | | | | | | | | | | | | |
| | 45 | | 10 | 84 | | 10 | | | | | | |
| 25 | | | | | | | | SM | Brown silty fine to medium sand interlayered with silty gravel and coarse sand (dense, wet) | | | |
| | 50 | | 12 | 34 | | 11 | | | | | | |



## Log of Boring B-1 (continued)

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

| Drilled | Start 11/1/2018 | End 11/1/2018 | Total Depth (ft) 81.5 | Logged By WCW | Checked By MSH | Driller Advanced Drill Technologies | Drilling Method Hollow-stem Auger |
|---|---|---|---|---|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| Surface Elevation (ft)<br>Vertical Datum | 73<br>NAVD88 | Hammer<br>Data | Autohammer<br>140 (lbs) / 30 (in) Drop | Drilling<br>Equipment | Diedrich D50 Turbo |
| Latitude<br>Longitude | 47.54206<br>-122.03464 | System<br>Datum | WA State Plane North<br>NAD83 (feet) | See "Remarks" section for groundwater observed | |

Notes:

| Elevation (feet) | Depth (feet) | FIELD DATA: Interval / Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | SM | Brown-gray silty fine to coarse sand with gravel (loose, moist) (fill) | | | |
| 70 | | 12 | 8 | | 1 MC | | | | 9 | | |
| | 5 | 10 | 6 | | 2a 2b | | | Becomes dark brown | | | |
| 65 | | | | | | | GP-GM | Brown fine to coarse gravel with silt and sand (medium dense, wet) | | | |
| | 10 | 12 | 15 | | 3 | | | | | | Groundwater observed at approximately 9 feet at time of drilling |
| 60 | | | | | | | SM | Dark gray silty fine to coarse sand with gravel (medium dense, wet) | | | |
| | 15 | 6 | 16 | | 4 | | | | | | |
| 55 | | | | | | | OL | Dark brown organic silt with sand (soft, wet) | | | |
| | 20 | 18 | 3 | | 5a MC<br>5b MC | | | | 56<br>23 | | Driller added mud |
| | | | | | | | SM | Silty fine to medium sand with occasional gravel (very loose, wet) | | | |
| 50 | | | | | | | | | | | |
| | 25 | 13 | 33 | | 6a 6b | | GP-GM | Brown fine to coarse gravel with silt and sand (dense, wet) | | | |
| 45 | | | | | | | SM | Brown silty fine to coarse sand with gravel (loose to medium dense, wet) | | | |
| | 30 | 18 | 9 | | 7 %F | | | | 17 | 15 | |
| 40 | | | | | | | | | | | |
| | 35 | | | | | | | | | | |

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

## Log of Boring B-2

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-3
Sheet 1 of 3

| Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| 35 | X | 7 | 16 | | 8 | | | | | |
| | | | | | | GM | Brown silty gravel with sand (medium dense, wet) | | | |
| 40 | X | 12 | 14 | | 9a<br>9b | | | | | |
| | | | | | | SM | Brown silty fine to coarse sand with gravel (medium dense, wet) | | | |
| | | | | | | GP-GM | Brown fine to coarse gravel with silt and sand (dense, wet) | | | |
| 45 | X | 12 | 35 | | 10 | | | | | |
| 50 | X | 12 | 48 | | 11a<br>11b | | | | | |
| | | | | | | SP-SM | Brown fine to coarse sand with silt and gravel (medium dense, wet) | | | |
| 55 | X | 12 | 28 | | 12<br>%F | | | 10 | 7 | |
| 60 | X | 12 | 10 | | 13 | | | | | |
| | | | | | | SM/OL | Gray silty fine sand interlayered with gray organic silt and trace wood debris (medium dense, moist) | | | |
| 65 | X | 18 | 13 | | 14<br>MC | | | 37 | | |
| | | | | | | SM | Dark gray silty fine sand (medium dense, wet) | | | |
| 70 | X | 18 | 23 | | 15<br>MC | | | 27 | | |
| 75 | X | 18 | 22 | | 16 | | Oxidation staining | | | |

Elevation (feet): 35, 30, 25, 20, 15, 10, 5, 0



## Log of Boring B-2 (continued)

| Elevation (feet) | Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name Testing | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -5 | | | | | | | | SM | Blue-gray silty fine sand (dense, moist) | | | |
| | 80 | | 18 | 39 | | 17a 17b | | | | | | |
| | | | | | | | | GM | Brown-gray silty fine to medium gravel with sand (dense, moist) | | | |

## Log of Boring B-2 (continued)

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-3
Sheet 3 of 3

| | | | | | |
|---|---|---|---|---|---|
| Drilled | Start 11/2/2018 | End 11/2/2018 | Total Depth (ft) 31.5 | Logged By WCW / Checked By MSH | Driller Advanced Drill Technologies |
| Drilling Method | Hollow-stem Auger | | | | |
| Hammer Data | Autohammer 140 (lbs) / 30 (in) Drop | Drilling Equipment | Diedrich D50 Turbo | | |
| Surface Elevation (ft) / Vertical Datum | 72 / NAVD88 | Top of Casing Elevation (ft) | | | |
| Latitude / Longitude | 47.5419 / -122.03401 | Horizontal Datum | WA State Plane North NAD83 (feet) | | |

A 2-in well was installed on 11/2/2018 to a depth of 14 ft.

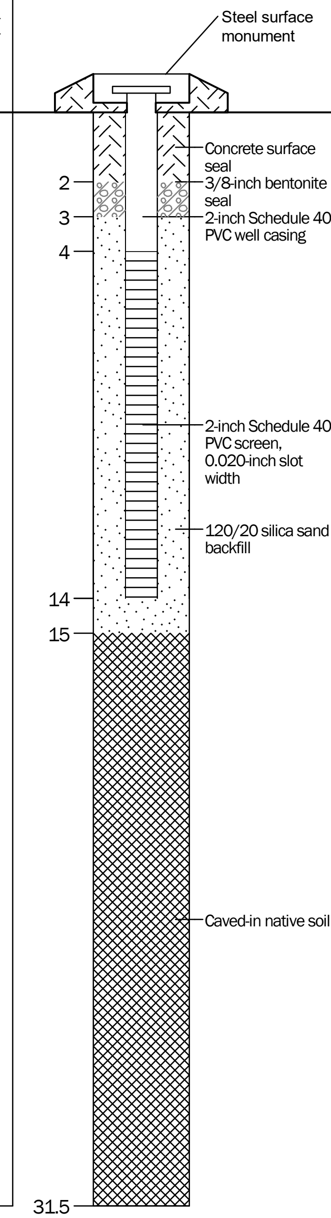
| Groundwater Date Measured | Depth to Water (ft) | Elevation (ft) |
|---|---|---|
| 1/15/2019 | 6.50 | 65.50 |

Notes:

| Elevation (feet) | Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Water Level | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 72 | 0 | | | | | | | SM | Brown-tan silty fine to coarse sand (medium dense, moist) (fill) | | |
| | | X | 18 | 14 | | 1 / MC | | | | 17 | |
| | 5 | X | 10 | 16 | | 2 | ▼ | | Grades with gravel | | |
| | | | | | | | | SP-SM | Blue-gray fine to coarse sand with silt and gravel (medium dense, moist) | | |
| | 10 | X | 8 | 17 | | 3 | | | | | |
| | | | | | | | | SM | Blue-gray silty fine to coarse sand with gravel (loose, wet) | | |
| | 15 | X | 8 | 9 | | 4 | | | | | |
| | | | | | | | | GP-GM | Blue-gray fine to coarse gravel with silt and sand (dense, wet) | | |
| | 20 | X | 10 | 33 | | 5 | | | | | |
| | | | | | | | | SP-SM | Bluish gray fine to coarse sand with silt and gravel (very dense, wet) | | |
| | 25 | X | 8 | 50/5" | | 6 | | | | | |
| | | | | | | | | SP | Gray fine to medium sand (very dense, wet) | | |
| | 30 | X | 10 | 57 | | 7a / 7b | | GP-GM | Tan-brown fine to coarse gravel with silt and sand (very dense, wet) | | |

Elevation scale: 70, 65, 60, 55, 50, 45

## WELL LOG




Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Date:1/17/19 Path:P:\23\23589001\GINT\2358900100.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_WELL_%F

# Log of Monitoring Well MW-1

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-4
Sheet 1 of 1

| Drilled | Start 11/2/2018 | End 11/2/2018 | Total Depth (ft) | 31.5 | Logged By WCW, Checked By MSH | Driller | Advanced Drill Technologies | Drilling Method | Hollow-stem Auger |
|---|---|---|---|---|---|---|---|---|---|

| | | | |
|---|---|---|---|
| Hammer Data | Autohammer 140 (lbs) / 30 (in) Drop | Drilling Equipment | Diedrich D50 Turbo |
| Surface Elevation (ft) Vertical Datum | 73 NAVD88 | Top of Casing Elevation (ft) | |
| Latitude Longitude | 47.546 -122.03427 | Horizontal Datum | WA State Plane North NAD83 (feet) |

A 2-in well was installed on 11/2/2018 to a depth of 15 ft.

| Groundwater Date Measured | Depth to Water (ft) | Elevation (ft) |
|---|---|---|
| 1/15/2019 | 7.85 | 65.15 |

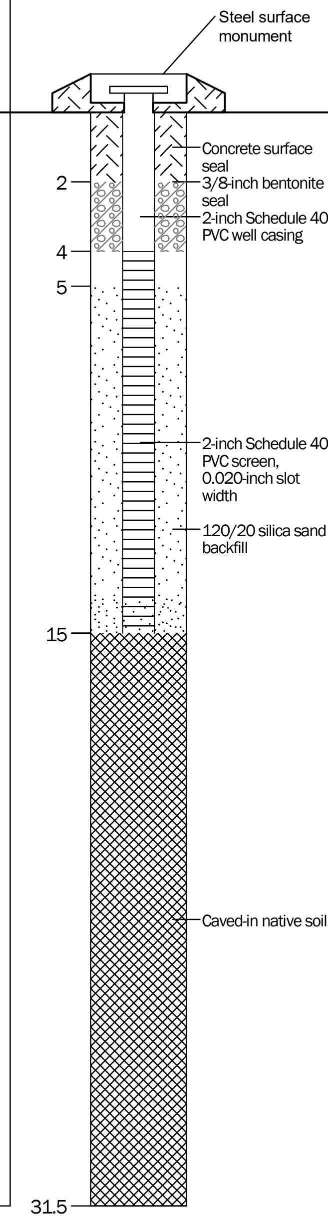
Notes:

## FIELD DATA

| Depth (feet) | Recovered (in) | Blows/foot | Sample Name Testing | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) |
|---|---|---|---|---|---|---|---|
| 0 | | | | SM | Brown silty fine to coarse sand with gravel and trace organic matter (medium dense, moist) (fill) | | |
| | 18 | 18 | 1 %F | | | 17 | 40 |
| 4 | | | | SP-SM | Brown fine to coarse sand with silt, occasional gravel and wood debris (loose, moist) | | |
| 5 | 18 | 5 | 2 SA | | | 15 | 6 |
| | | | | ML | Blue-gray sandy silt, oxidation staining (soft, moist) | | |
| 10 | 14 | 3 | 3a | | | | |
| | | | 3b %F | SM | Blue-gray silty fine to medium sand with pockets of silt, oxidation staining (very loose, moist) | 34 | 30 |
| 15 | 16 | 29 | 4 | | Becomes medium dense and with gravel | | |
| | | | | SP-SM | Brown-gray fine to coarse sand with silt and occasional gravel (dense, wet) | | |
| 20 | 16 | 33 | 5 | | | | |
| 25 | 10 | 50/4" | 6 | | Becomes very dense | | |
| | | | | SP | Brown-gray fine to medium sand (very dense, wet) | | |
| 30 | 14 | 50 | 7 | | | | |

Elevation (feet): 70, 65, 60, 55, 50, 45

## WELL LOG




Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.



# Log of Monitoring Well MW-2

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-5
Sheet 1 of 1

| Drilled | Start | End | Total Depth (ft) | Logged By | Checked By | Driller | Drilling Method |
|---|---|---|---|---|---|---|---|
| | 11/2/2018 | 11/2/2018 | 46.5 | WCW | MSH | Advanced Drill Technologies | Hollow-stem Auger |

| | | | |
|---|---|---|---|
| Hammer Data | Autohammer 140 (lbs) / 30 (in) Drop | Drilling Equipment | Diedrich D50 Turbo |
| Surface Elevation (ft) Vertical Datum | 73 NAVD88 | Top of Casing Elevation (ft) | |
| Latitude Longitude | 47.54237 -122.03454 | Horizontal Datum | WA State Plane North NAD83 (feet) |

A 2-in well was installed on 11/2/2018 to a depth of 15 ft.

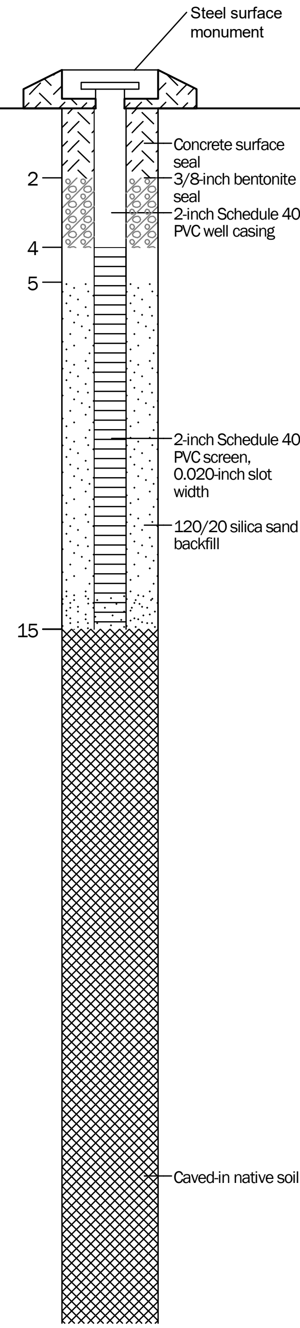
| Groundwater Date Measured | Depth to Water (ft) | Elevation (ft) |
|---|---|---|
| 1/15/2019 | 7.05 | 65.95 |

Notes:

FIELD DATA

| Elevation (feet) | Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | | | | | SM | Brown to blue silty fine to coarse sand with gravel, and trace organic matter (medium dense, moist) (fill) | | |
| 70 | | | 16 | 18 | | 1 | | | | | | |
| | 5 | | 16 | 8 | | 2 SA | | | SM | Brown silty fine sand with gravel, oxidation staining and organic matter (stiff, moist) | 34 | 30 |
| 65 | | | | | | | ▼ | | | | | |
| | 10 | | 0 | 2 | | NR | | | | | | |
| 60 | | | | | | | | | SP-SM | Blue gray fine to coarse sand with silt, gravel and occasional cobbles (loose to medium dense, wet) | | |
| | 15 | | 6 | 19 | | 3 | | | | | | |
| 55 | | | | | | | | | | | | |
| | 20 | | 12 | 8 | | 4 %F | | | | | 18 | 11 |
| 50 | | | | | | | | | | | | |
| | 25 | | 12 | 33 | | 5 | | | | | | |
| 45 | | | | | | | | | | | | |
| | 30 | | 14 | 14 | | 6 | | | | | | |
| 40 | | | | | | | | | | | | |
| | 35 | | | | | | | | | | | |

WELL LOG




Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Date:1/17/19 Path:P:\23\23589001\GINT\2358900100.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_WELL_%F

## Log of Monitoring Well MW-3

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-6
Sheet 1 of 2

| Elevation (feet) | Depth (feet) | Interval | Recovered (in) | Blows/foot | Collected Sample | Sample Name / Testing | Water Level | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | WELL LOG |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 35 | | 3 | 12 | | 7 | | | | Grades with occasional gravel | | | Caved-in native soil |
| 35 | | | | | | | | | SP-SM | Brown fine to medium sand with silt (medium dense, wet) | | | |
| | 40 | | 18 | 18 | | 8 | | | | | | | |
| 30 | | | | | | | | | | | | | |
| | 45 | | 18 | 42 | | 9 | | | | Becomes dense | | | 46.5 |



## Log of Monitoring Well MW-3 (continued)

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-6
Sheet 2 of 2

| | | | | | |
|---|---|---|---|---|---|
| Date Excavated | 10/31/2018 | Total Depth (ft) | 11 | Logged By WCW<br>Checked By MSH | Excavator<br>Equipment Takeuchi TB260 Excavator |

See "Remarks" section for groundwater observed
See "Remarks" section for caving observed

| | | | | | |
|---|---|---|---|---|---|
| Surface Elevation (ft)<br>Vertical Datum | 72<br>NAVD88 | Latitude<br>Longitude | 47.54245<br>-122.03392 | Coordinate System<br>Horizontal Datum | WA State Plane North<br>NAD83 (feet) |

| Elevation (feet) | Depth (feet) | SAMPLE: Testing Sample | SAMPLE: Sample Name / Testing | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| 72 | 0 | | | | TS | Grass and tree roots | | | |
| | | ▯ | 1<br>MC | | SM | Brown silty fine to medium sand with organic matter (loose, moist) (fill) | 18 | | Probe depth 8 to 9 inches<br>Large roots from nearby tree |
| 71 | 1 | | | | SM | Brown silty fine to coarse sand with gravel and trace organic matter (loose to medium dense, moist) | | | |
| 70 | 2 | ▯ | 2 | | | | | | |
| | | | | | | With oxidation staining | | | Probe depth 3 to 5 inches |
| 69 | 3 | ▯ | 3 | | | | | | |
| 68 | 4 | | | | | | | | Moderate caving observed at approximately 4½ feet |
| 67 | 5 | | | | SM | Brown silty fine to medium sand with gravel (medium dense, moist) | | | |
| 66 | 6 | ▯ | 4<br>MC | | | | 26 | | |
| 65 | 7 | | | | | Grades with occasional cobbles | | | |
| 64 | 8 | | | | | | | | Slight groundwater seepage observed at approximately 8 feet |
| | | ▯ | 5 | | | | | | |
| 63 | 9 | ▯ | 6<br>MC | | SM | Dark brown silty fine to coarse sand with gravel and organic matter (loose to medium dense, wet) | 27 | | |
| | | | | | SP-SM | Brown fine to medium sand with silt and occasional gravel (medium dense, moist) | | | |
| 62 | 10 | ▯ | 7 | | | | | | |
| 61 | 11 | | | | | | | | |

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

# Log of Test Pit TP-1

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

| Date Excavated | 10/31/2018 | Total Depth (ft) | 10.5 | Logged By | WCW | Excavator | | See "Remarks" section for groundwater observed |
|---|---|---|---|---|---|---|---|---|
| | | | | Checked By | MSH | Equipment | Takeuchi TB260 Excavator | See "Remarks" section for caving observed |

| Surface Elevation (ft) | 72 | Latitude | 47.54205 | Coordinate System | WA State Plane North |
|---|---|---|---|---|---|
| Vertical Datum | NAVD88 | Longitude | -122.03374 | Horizontal Datum | NAD83 (feet) |

| Elevation (feet) | Depth (feet) | SAMPLE: Sample Name / Testing | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|
| 72 | 0 | | TS | Brown silty fine to medium sand with gravel (topsoil) | | | Probe depth 6 to 8 inches |
| 71 | 1 | 1 / MC | SM | Tan-brown silty fine to coarse sand with gravel and occasional cobbles (dense, moist) (fill) | 7 | | |
| 70 | 2 | 2 / MC | SM | Gray silty fine to medium sand with occasional gravel (medium dense, moist) | 16 | | |
| 69 | 3 | 3 | SM | Dark brown silty fine to medium sand with gravel and organic matter (medium dense, moist) | | | |
| 68 | 4 | 4 | GP-GM | Brown fine gravel with silt and sand (medium dense, moist) | | | Probe depth 1 to 3 inches |
| 67 | 5 | 5 / SA | GW-GM | Brown silty fine to coarse gravel with silt, sand and organic matter (medium dense, moist) | 7 | 10 | |
| 66 | 6 | 6 / MC | SM | Dark gray silty fine to coarse sand with gravel (loose, moist to wet) | 12 | | Minor caving observed at approximately 6 feet |
| 65 | 7 | 7 | SM | Brown silty fine to medium sand (loose to medium dense, moist) | | | Slight groundwater seepage observed at approximately 7 feet |
| 64 | 8 | | | | | | |
| 63 | 9 | 8 | SM | Dark gray silty fine sand (medium dense, moist) | | | |
| 62 | 10 | 9 | ML | Dark gray-brown sandy silt with gravel (stiff, moist) | | | |

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

# Log of Test Pit TP-2

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-8
Sheet 1 of 1

| Date Excavated | 10/31/2018 | Total Depth (ft) | 10.5 | Logged By | WCW | Excavator Equipment | Takeuchi TB260 Excavator | See "Remarks" section for groundwater observed |
|---|---|---|---|---|---|---|---|---|
| | | | | Checked By | MSH | | | Caving not observed |

| Surface Elevation (ft) | 71 | Latitude | 47.5416 | Coordinate System | WA State Plane North |
|---|---|---|---|---|---|
| Vertical Datum | NAVD88 | Longitude | -12.03383 | Horizontal Datum | NAD83 (feet) |

| Elevation (feet) | Depth (feet) | SAMPLE: Sample Name / Testing | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|
| | 0 | | TS | Brown silty fine sand with trace organic matter (loose, moist) (fill) | | | Probe depth 1 to 2 inches |
| 70 | 0.5 | | ML | Gray silt with sand and trace organic matter (medium stiff, moist) | | | |
| 69–68 | 1–3 | 1, 2 | SM | Brown silty fine to coarse sand with gravel (medium dense, moist) | | | |
| 67 | 3–4 | 3 | SM | Black-gray silty fine sand (loose to medium dense, moist) | | | Probe depth 1 to 2 inches |
| 66 | 4–5 | 4 | SM | Gray silty fine to medium sand with occasional gravel (loose, moist) | | | |
| 65 | 5–6.5 | 5 MC | OL | Dark brown-black organic silt (soft, moist) | 67 | | |
| 64–61 | 6.5–10 | 6 | SP-SM | Gray fine to coarse sand with silt and gravel (loose to medium dense, moist to wet) | | | Slight groundwater seepage observed at 8½ feet |
| | 10–10.5 | 7 | SP-SM | Gray fine to medium sand with silt (loose to medium dense, wet) | | | |

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

## Log of Test Pit TP-3

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-9
Sheet 1 of 1

| Date Excavated | 10/31/2018 | Total Depth (ft) | 5.5 | Logged By WCW<br>Checked By MSH | Excavator<br>Equipment Takeuchi TB260 Excavator | Groundwater not observed<br>Caving not observed |
|---|---|---|---|---|---|---|

| Surface Elevation (ft) | 72 | Latitude | 47.54207 | Coordinate System | WA State Plane North |
|---|---|---|---|---|---|
| Vertical Datum | NAVD88 | Longitude | -122.03503 | Horizontal Datum | NAD83 (feet) |

| Elevation (feet) | Depth (feet) | SAMPLE: Testing Sample | SAMPLE: Sample Name / Testing | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | TS | 3-4 inch root mat | | | |
| | | | | | SM | Brown silty fine to coarse sand with gravel (medium dense, moist) (fill) | | | Probe depth 3 to 6 inches |
| 71 | 1 | | 1 / SA | | | | 14 | 22 | |
| 70 | 2 | | | | | | | | Probe depth 2 to 3 inches |
| 69 | 3 | | | | | | | | |
| 68 | 4 | | | | ML | Dark gray-black silt with organic matter (soft, moist) | | | |
| 67 | 5 | | 2 / SA | | SP-SM | Dark gray fine to coarse sand with silt and occasional gravel (loose to medium dense, moist) | 17 | 5 | |

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

## Log of Test Pit TP-4

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

| | | | | | |
|---|---|---|---|---|---|
| Date Excavated | 10/31/2018 | Total Depth (ft) | 10.5 | Logged By WCW<br>Checked By MSH | Excavator Equipment Takeuchi TB260 Excavator | See "Remarks" section for groundwater observed<br>See "Remarks" section for caving observed |

| | | | | | |
|---|---|---|---|---|---|
| Surface Elevation (ft)<br>Vertical Datum | 73<br>NAVD88 | Latitude<br>Longitude | 47.5426<br>-122.0342 | Coordinate System<br>Horizontal Datum | WA State Plane North<br>NAD83 (feet) |

| Elevation (feet) | Depth (feet) | SAMPLE: Testing Sample | SAMPLE: Sample Name / Testing | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | | | TS | Dark brown silty fine to medium sand with gravel and grass roots (loose, wet) (topsoil) | | | |
| 72 | 1 | | 1 MC | | SM | Gray silty fine to coarse sand with gravel (medium dense, moist) (fill) | 7 | | |
| | | | | | SM | Brown silty fine sand with gravel (loose, moist) | | | |
| 71 | 2 | | 2 | | | | | | |
| | | | 3 SA | | GW-GM | Reddish brown fine to coarse gravel with silt, sand and occasional cobbles (loose, moist) | 6 | 6 | Probe depth 6 to 8 inches |
| 70 | 3 | | | | | | | | Minor caving observed from 3 to 6 feet |
| 69 | 4 | | | | SM | Tan-brown silty fine to medium sand with gravel (loose, moist) | | | Probe depth 12 to 14 inches |
| 68 | 5 | | | | | | | | |
| 67 | 6 | | 4 | | | | | | |
| 66 | 7 | | 5 | | SM | Reddish brown silty fine sand (loose, wet) | | | Slight groundwater seepage observed at approximately 7½ feet |
| 65 | 8 | | | | ML | Gray silt with occasional sand (soft, wet) | | | |
| | | | | | GP-GM | Reddish brown fine to coarse gravel with silt and sand (medium dense, wet) | | | |
| 64 | 9 | | 6 | | | | | | Moderate groundwater seepage observed at approximately 9 feet<br>Moderate caving observed at approximately 9 feet |
| 63 | 10 | | 7 | | | | | | |

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.



## Log of Test Pit TP-5

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

| | | | | | |
|---|---|---|---|---|---|
| Date Excavated | 10/31/2018 | Total Depth (ft) | 5 | Logged By WCW<br>Checked By MSH | Excavator<br>Equipment Takeuchi TB260 Excavator | Groundwater not observed<br>Caving not observed |

| | | | |
|---|---|---|---|
| Surface Elevation (ft)<br>Vertical Datum | 74<br>NAVD88 | Latitude<br>Longitude | 47.54237<br>-122.03521 | Coordinate System<br>Horizontal Datum | WA State Plane North<br>NAD83 (feet) |

| Elevation (feet) | Depth (feet) | SAMPLE: Testing Sample | SAMPLE: Sample Name / Testing | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| 74 | 0 | | | | TS | 3-4 inch root mat | | | |
| 73 | 1 | | 1<br>MC | | SP-SM | Tan-brown fine to medium sand with silt and occasional gravel (loose to medium dense, moist) (fill) | 11 | | Probe depth 3 to 4 inches |
| 72 | 2 | | | | SM | Blue-gray silty fine to coarse sand with gravel (medium dense, moist) | | | Probe depth ½ inch |
| 71 | 3 | | 2 | | | | | | |
| 70 | 4 | | | | | | | | |
| 69 | 5 | | | | | | | | |

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

# Log of Test Pit TP-6

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

| Date Excavated | 10/31/2018 | Total Depth (ft) | 7.5 | Logged By | WCW | Excavator | | Groundwater not observed |
|---|---|---|---|---|---|---|---|---|
| | | | | Checked By | MSH | Equipment | Takeuchi TB260 Excavator | Caving not observed |

| Surface Elevation (ft) | 73 | Latitude | 47.54177 | Coordinate System | WA State Plane North |
|---|---|---|---|---|---|
| Vertical Datum | NAVD88 | Longitude | -122.03454 | Horizontal Datum | NAD83 (feet) |

| Elevation (feet) | Depth (feet) | Testing Sample | Sample Name Testing | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | TS | Dark brown silty fine to medium sand (loose, moist) (topsoil) | | | Probe depth 2 to 3 inches |
| 72 | 1 | | 1 MC | | ML | Tan-brown sandy silt with gravel (stiff, moist) (fill) | 18 | | |
| 71 | 2 | | 2 MC | | SM | Blue-gray silty fine to coarse sand with occasional gravel (medium dense, moist) | 18 | | Probed depth ½ to 1 inch |
| 70 | 3 | | | | | | | | |
| 69 | 4 | | 3 | | | | | | |
| 68 | 5 | | | | | | | | |
| 67 | 6 | | 4 MC | | PT | Dark brown organic silt and peat with sand (soft, moist) | 76 | | |
| 66 | 7 | | | | | | | | |
| | | | | | SM | Dark gray-brown silty fine to coarse sand with gravel and cobbles (medium dense, moist) | | | |

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.



# Log of Test Pit TP-7

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

| Date Excavated | 10/31/2018 | Total Depth (ft) | 8.5 | Logged By | WCW | Excavator | | Groundwater not observed |
|---|---|---|---|---|---|---|---|---|
| | | | | Checked By | MSH | Equipment | Takeuchi TB260 Excavator | Caving not observed |

| Surface Elevation (ft) | 73 | Latitude | 47.54232 | Coordinate System | WA State Plane North |
|---|---|---|---|---|---|
| Vertical Datum | NAVD88 | Longitude | -122.03484 | Horizontal Datum | NAD83 (feet) |

| Elevation (feet) | Depth (feet) | SAMPLE: Testing Sample | SAMPLE: Sample Name / Testing | Graphic Log | Group Classification | MATERIAL DESCRIPTION | Moisture Content (%) | Fines Content (%) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| 73 | 0 | | | | TS | 3-4 inch root mat | | | |
| | | | | | SM | Brown silty fine to coarse sand with gravel and occasional cobbles (loose to medium dense, moist) (fill) | | | Probe depth 3 to 5 inches |
| 72 | 1 | | 1 MC | | | | 18 | | |
| | | | | | SM | Dark gray-brown silty fine to coarse sand with gravel (medium dense, moist) | | | |
| 71 | 2 | | 2 SA | | | | 10 | 17 | Probe depth ½ inch |
| 70 | 3 | | | | | | | | |
| | | | | | | | | | Probe depth 3 to 5 inches |
| 69 | 4 | | | | | | | | |
| 68 | 5 | | | | | | | | |
| 67 | 6 | | | | | | | | |
| 66 | 7 | | | | | | | | |
| 65 | 8 | | | | | | | | |

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

## Log of Test Pit TP-8

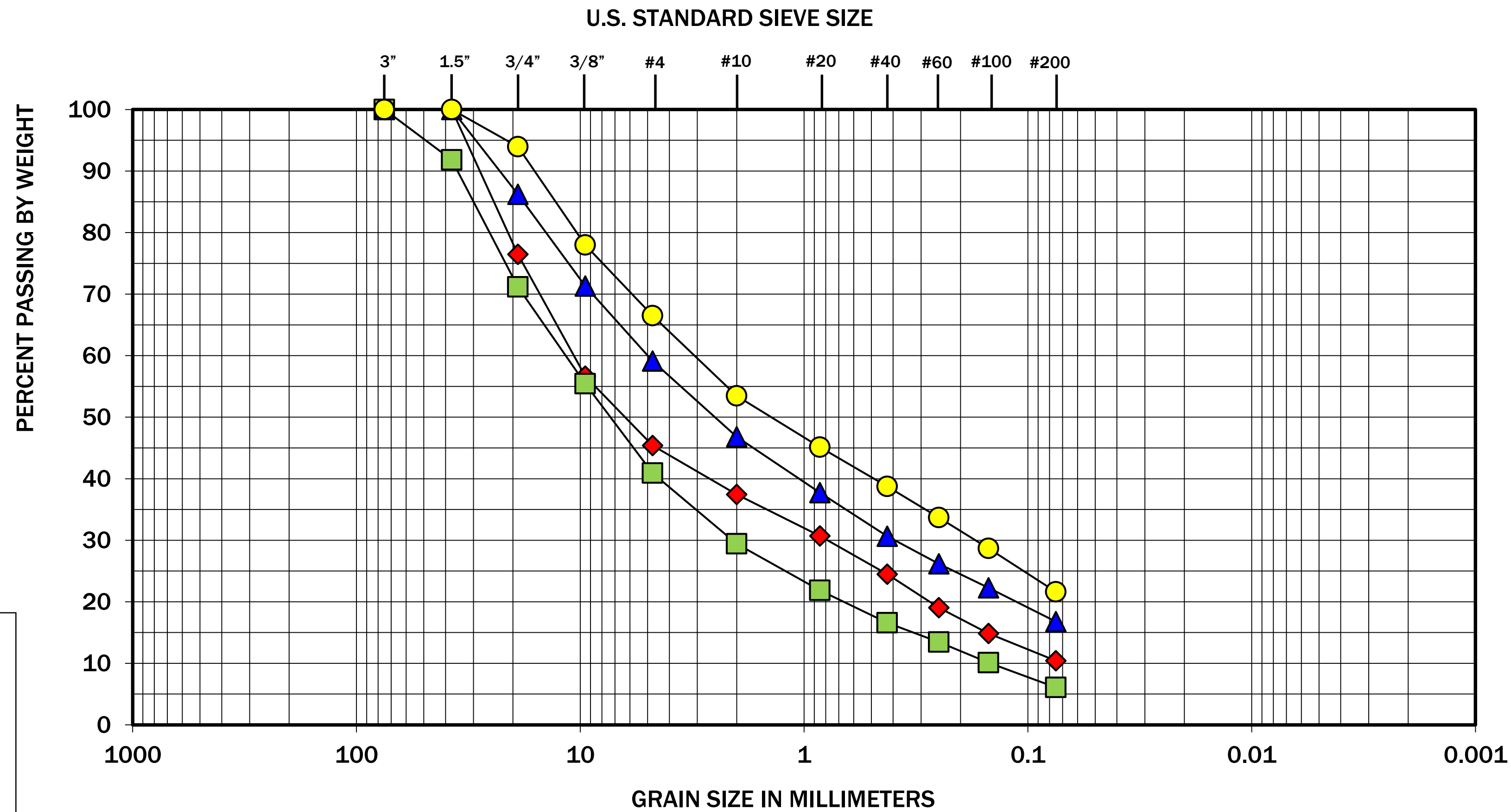
GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-14
Sheet 1 of 1

Date:1/17/19 Path:P:\23\23589001\GINT\2358900100.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_TESTPIT_1P_GEOTEC_%F

23589-001-00 Date Exported: 11/14/18




<table>
<tr><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

| Symbol | Boring Number | Depth (feet) | Moisture (%) | Soil Description |
|---|---|---|---|---|
| ◆ | TP-2 | 5 | 7 | Fine to coarse gravel with silt and sand (GP-GM) |
| ■ | TP-5 | 3 | 6 | Fine to coarse gravel with silt and sand (GW-GM) |
| ▲ | TP-8 | 2 | 10 | Silty fine to coarse sand with gravel (SM) |
| ● | TP-4 | 1 | 14 | Silty fine to coarse sand with gravel (SM) |



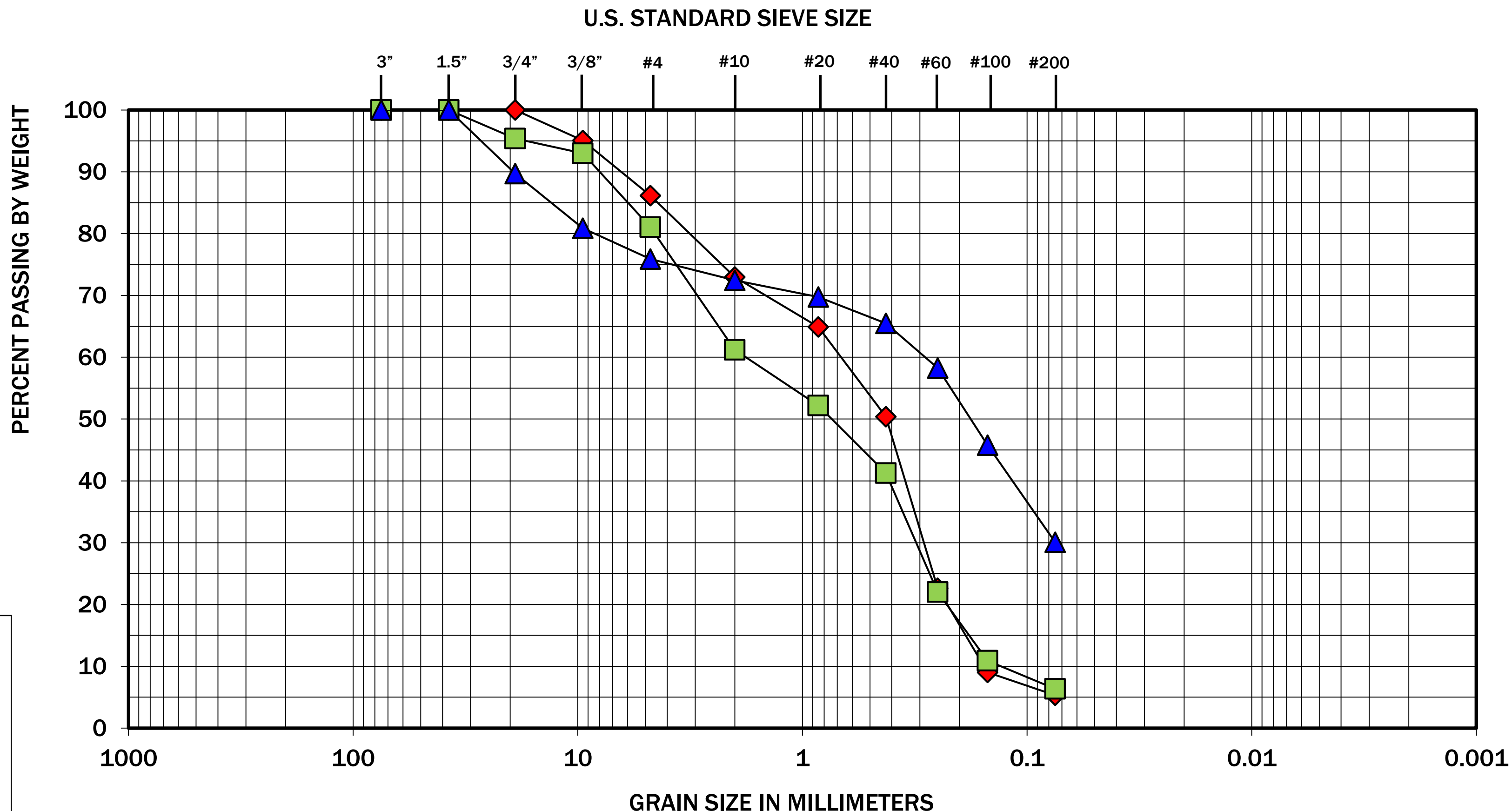
The grain size analysis results were obtained in general accordance with ASTM D 6913. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

## Sieve Analysis Results

Evergreen Ford Lincoln
Issaquah, Washington

GeoEngineers

**Figure A-15**




| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | COARSE | FINE | COARSE | MEDIUM | FINE | |

| Symbol | Boring Number | Depth (feet) | Moisture (%) | Soil Description |
|---|---|---|---|---|
| ◆ | TP-4 | 5 | 17 | Fine to coarse sand with silt and occasional gravel (SP-SM) |
| ■ | MW-2 | 5 | 15 | Fine to coarse sand with silt and gravel (SP-SM) |
| ▲ | MW-3 | 5 | 34 | Silty fine sand with gravel (SM) |

Note: This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc. Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes.

The grain size analysis results were obtained in general accordance with ASTM D 6913. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

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Sieve Analysis Results

Evergreen Ford Lincoln
Issaquah, Washington

Figure A-16

# APPENDIX B
## Previous Explorations

## APPENDIX B
## PREVIOUS EXPLORATIONS

Included in this section are logs from previous studies completed in the immediate vicinity of the project site.

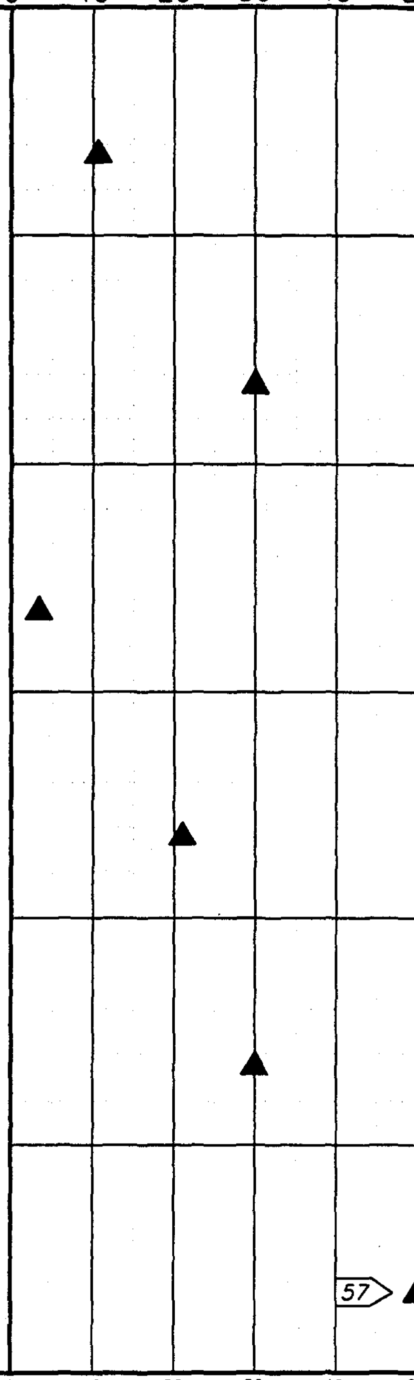
- The log of one boring (B-1) completed by AGRA Earth & Environmental in 1997 for the AT&T Cellular Tower Site

PROJECT: Issaquah AT&T Cellular Tower Site

W.O. 7-91M-11507-0 BORING No. B-1



| DEPTH (FEET) | SOIL DESCRIPTION<br>Approximate ground surface elevation: *Unknown* | SAMPLE TYPE | SAMPLE NUMBER | GROUND WATER | STANDARD PENETRATION RESISTANCE ▲ Blows per foot (0 10 20 30 40 50) | TESTING |
|---|---|---|---|---|---|---|
| 0 | *Soft grading to stiff, wet, brown SILT with trace sand (Fill)* | | | | | |
| | | I | *S-1* | | | |
| 5 | | | | | | |
| | *Medium dense, wet, brown, silty, gravelly SAND* | | | | | |
| | | I | *S-2* | | | |
| 10 | | | | | | |
| | | | | ▼ ATD | | |
| | *Soft, wet to saturated, gray mottled, SILT* | I | *S-3* | | | |
| 15 | | | | | | |
| | *Medium dense, wet, brown, silty, gravelly SAND* | | | | | |
| | | I | *S-4* | | | |
| 20 | | | | | | |
| | | I | *S-5* | | | |
| 25 | | | | | | |
| | *Very dense, wet, brown, silty SAND* | I | *S-6* | | 57 | |
| | *Very dense, wet, brown, silty, sandy GRAVEL* | | | | | |
| 30 | | | | | | |




*(continued)*

**LEGEND**

I 2-inch OD split-spoon sample

▼ ATD Groundwater level at time of drilling

0 10 20 30 40 50
MOISTURE CONTENT
Plastic limit — Natural — Liquid limit

AGRA
Earth & Environmental
11335 N.E. 122nd Way, Suite 100
Kirkland, WA, U.S.A. 98034-6918



Drilling started: *14 February 1997* Drilling completed: *14 February 1997* Logged by: *JRZ*

PROJECT: Issaquah AT&T Cellular Tower Site W.O. 7-91M-11507-0 BORING No. B-1

| DEPTH (FEET) | SOIL DESCRIPTION<br>Approximate ground surface elevation: *Unknown* | SAMPLE TYPE | SAMPLE NUMBER | GROUND WATER | STANDARD PENETRATION RESISTANCE ▲ Blows per foot (0 10 20 30 40 50) | Page 2 of 2 TESTING |
|---|---|---|---|---|---|---|
| 30 | *Silty, sandy GRAVEL as above* | | | | | |
| | *Very dense, wet, brown SAND* | I | S-7A | | | |
| | | I | S-7B | | 60 ▲ | |
| 35 | *Boring terminated at approximately 34 feet* | | | | | |
| 40 | | | | | | |
| 45 | | | | | | |
| 50 | | | | | | |
| 55 | | | | | | |
| 60 | | | | | | |

MOISTURE CONTENT (0 10 20 30 40 50)

Plastic limit — Natural — Liquid limit

**LEGEND**

I 2-inch OD split-spoon sample

ATD Groundwater level at time of drilling

AGRA
**Earth & Environmental**
11335 N.E. 122nd Way, Suite 100
Kirkland, WA, U.S.A. 98034-6918



Drilling started: *14 February 1997* Drilling completed: *14 February 1997* Logged by: *JRZ*

# APPENDIX C
## Report Limitations and Guidelines for Use

## APPENDIX C
## REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

### Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of Strotkamp Associates and project team members for the Evergreen Ford Lincoln property located in Issaquah, Washington. This report may be made available to prospective contractors for their bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with which there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

### A Geotechnical Engineering or Geologic Report Is Based on a Unique Set of Project-Specific Factors

This report has been prepared for the Evergreen Ford Lincoln property in Issaquah, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;

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[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

- composition of the design team; or
- project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

### Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

### Most Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

### Geotechnical Engineering Report Recommendations Are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

### A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

### Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

### Give Contractors a Complete Report and Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

### Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor’s procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

### Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory “limitations” provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these “Report Limitations and Guidelines for Use” apply to your project or site.

### Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

## Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants and no conclusions or inferences should be drawn regarding Biological Pollutants, as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.

GeoEngineers

GeoEngineers

17425 NE Union Hill Road, Suite 250
Redmond, Washington 98052
425.861.8000

May 30, 2019

Evergreen Ford Lincoln
c/o Strotkamp Architects
PO Box 501
Burlington, Washington 98233

Attention: Tom Strotkamp and David Estes

Subject: Hydrogeologic Services, Groundwater Mounding Study
Evergreen Ford Lincoln
22909 SE 66th Street
Issaquah, Washington
File No. 23589-001-00

## 1.0 INTRODUCTION

A groundwater mounding analysis has been developed to support the stormwater facilities design for site improvements planned at Evergreen Ford Lincoln at 22909 SE 66th Street in Issaquah. Washington. Geotechnical exploration of the site is described in our Geotechnical Engineering Services report dated January 18, 2019.

The relatively shallow depth to groundwater beneath the site limits the amount of vertical separation that can be achieved below infiltration facilities to less than 5 feet. GeoEngineers has therefore completed a groundwater mounding analysis for two conceptual stormwater infiltration facilities (NE and SW facilities) at the project site based on facility designs initially developed by SCJ Alliance. The proposed facility locations are shown in Drawing No. SD-01 by SCJ Alliance dated March 2019.

The mounding analysis has been conducted using the SEEP/W groundwater flow and seepage modeling software. The model accurately represents both unsaturated groundwater flow (above the water table) and saturated groundwater flow (below the water table), for specified material properties (hydraulic conductivity and moisture content/porosity) and boundary conditions (groundwater level and infiltration rate).

The groundwater mounding analysis is performed using the maximum 30-day portion of the continuous inflow hydrograph generated by SCJ Alliance for both conceptual infiltration facilities, based on design infiltration rates of 2 in/hr (NE facility) and 5.5 in/hr (SW facility). The basal areas of the facilities were 1800 square feet ($ft^2$) and 7067 $ft^2$, respectively. The design infiltration rates were selected by SCJ Alliance based on the results of sieve analyses as provided in Table 6 of our Geotechnical Engineering Services Report.

Groundwater levels at the site were measured in three monitoring wells installed during our geotechnical exploration. Water levels measured in January 2019 at the monitoring well locations, as shown on the boring longs in Appendix A of our Geotechnical Engineering Services Report, were interpolated to estimate the seasonal high groundwater elevation below the larger SW facility at 64.76 feet.

The maximum stormwater inflow is conservatively assumed to occur when the groundwater level is at the estimated seasonal high elevation and is routed into the facility assuming that the limiting saturated vertical hydraulic conductivity of the subgrade soils is 22 feet per day (ft/d). This is equivalent to a maximum infiltration rate of 11 in/hr, which represents our estimate of the uncorrected or unfactored saturated hydraulic conductivity of the subgrade soils at each facility.

## 2.0 GROUNDWATER MOUNDING ANALYSIS

The initial groundwater mounding analysis we performed showed that the facilities will function as designed for most storm durations and intensities but that significant mounding into each facility will occur during peak events. Based on our analysis, this indicated that facilities will likely back up under the highest intensity storms considered in the design time series, and that the facilities should be increased in size with reduced design infiltration rates. The latest design includes larger facilities and the groundwater mounding analysis was then repeated. Results for both analyses are included in this letter report.

### 2.1. Vertical Separation

The relatively shallow seasonal high groundwater elevation of 7 feet below ground surface (bgs) (64.76 feet elevation) anticipated at the Evergreen Ford Lincoln project site constrains the vertical separation that can be achieved below the proposed infiltration facilities. Systems have been designed to infiltrate stormwater at an elevation of 67.76 feet, giving a minimum vertical separation of 3 feet above the seasonal high groundwater table. Where the vertical separation is less than 5 feet, a mounding analysis must be accomplished to demonstrate that groundwater levels will not rise too high during major storm events and cause the stormwater infiltration facility to back-up and overflow.

### 2.2. Methodology

The groundwater mounding analysis has been conducted for both of the proposed stormwater infiltration facilities currently planned for the Evergreen Ford Lincoln project site. It has been accomplished using Version 8.16.1.13452 of the groundwater seepage modeling software, SEEP/W, which is part of the GeoStudio 2016 suite of specialist geotechnical software programs published by Geo-Slope Inc.

#### 2.2.1. Model Domain

SEEP/W is a powerful finite element groundwater flow simulation program that is well suited to performing mounding analyses. The analysis is conducted by developing a seepage profile that represents a vertical cross section through the subsurface soils at the midpoint of the infiltration facility. The model domain developed to represent both the NE and SW infiltration facilities proposed for the Evergreen Ford Lincoln improvements project is shown in Figure A, SeepW Model Construction and Materials and includes the following features:

- The main soil layers encountered in the borings are represented in the model as: silty sand with gravel, sand, sand with silt and gravel, gravel with silt and sand and silt with trace sand.
- The ground surface is represented by the green line at Elevation ±70.76 feet.
- Each infiltration facility is represented as a trench of drainage material excavated 4 feet deep to Elevation 67.76 feet.
- The full width of the infiltration trenches are 30 feet (NE facility) and 24 feet (SW facility) where infiltration will enter the subsurface soils at Elevation 67.76 feet.
- The bulk of the model domain represents subsurface soils extending laterally to each side of the infiltration trenches, to a maximum offset distance of 225 feet.
- Each soil layer is represented by a mesh of finite elements, which are assigned material properties and boundary conditions that allow the groundwater flow model to be solved mathematically. The mesh is finest around the infiltration facility, for increased accuracy where the highest hydraulic gradients and most rapidly changing groundwater conditions are expected.
- The initial water table is represented in the model at the seasonal high groundwater elevation of 64.76 feet, by applying a constant-head boundary condition at the side of the model domain.
- The base of the model extends to Elevation 25 feet, for a saturated aquifer thickness below the seasonal high-water table of 41.76 feet within the infiltration receptor; no flow is assumed to occur below this depth.

The model domain forms the basis for simulating the stormwater infiltration process and resulting mounding effect as the infiltrated water recharges the water table, creating a groundwater mound that extends above the seasonal high groundwater level. The model also simulates the dissipation of the groundwater mound as groundwater flows away from each facility during storm recession.

### 2.3. Stormwater Infiltration

Stormwater will be introduced to the infiltration facility via a series of Brentwood™ Modules, laid on a bed of gravel. The gravel is considerably more permeable than the native soils and is assumed to provide no restraint on the discharge of stormwater from the base of the modules. The SEEP/W model, therefore, applies stormwater discharge directly to the infiltration facility subgrade. The hydraulics of flow within the modules and through the gravel drain rock forming the backfill of the infiltration facility, are therefore not considered directly as part of the SEEP/W model, which is concerned primarily with build-up of the water table within the natural soils below the facility when stormwater is introduced.

### 2.4. Soil Hydraulic Properties

The subsurface soils at depth at the Evergreen Ford Lincoln site are varying layers of sand with silt and gravel below elevation 56 feet (approximately 15 feet below existing site grade and deeper). Within the surficial 15 feet, subsurface soils are largely the same but with occasional layers of silt. Sieve analyses were conducted for selected samples from the borings on site and the resulting grain-size distributions were used to estimate the permeability (or hydraulic conductivity) of the soils present at the site.

## 2.5. Hydraulic Conductivity

Groundwater flow and stormwater infiltration below each infiltration facility is controlled primarily by the hydraulic conductivity of each soil layer, which must be specified in the SEEP/W model. A publicly available spreadsheet tool, HydrogeoSieveXL, was used to estimate values of saturated hydraulic conductivities. HydrogeoSieveXL, developed by J. F. Devlin of the University of Kansas, is an Excel based computational program which uses sieve analysis data and 15 separate methods to estimate hydraulic conductivity values (Devlin 2015). Average values calculated using HydrogeoSieveXL for each of the available sieve analyses are listed in Table 1.

**TABLE 1. HYDRAULIC CONDUCTIVITY CALCULATED USING HYDROGEOSIEVEXL FROM GRAIN SIZE DISTRIBUTIONS**

| Borehole | Symbols | Units | MW-2 | MW-3 | TP-2 | TP-4 | TP-5 | TP-8 |
|---|---|---|---|---|---|---|---|---|
| Depth | | (ft) | 31.5 | 46.5 | 10.5 | 5.5 | 10.5 | 8.5 |
| Soil Layer[1] | | | SP-SM | SM | GW-GM | SM | GW-GM | SM |
| Grain Size Distribution | $D_{10}$<br>$D_{60}$<br>$D_{90}$ | (mm)<br>(mm)<br>(mm) | 0.13<br>1.78<br>7.99 | 0.03<br>0.28<br>19.35 | 0.075<br>10.68<br>28.09 | 0.023<br>3.08<br>16.00 | 0.15<br>11.61<br>35.33 | 0.03<br>4.99<br>22.89 |
| Fines Content | *Fines* | (%) | 6 | 30 | 10 | 22 | 6 | 2 |
| Calculated Hydraulic Conductivity | Log($K_s$)<br>$K_s$<br>$K_s$<br>$K_s$ | <br>(cm/s)<br>(ft/d)<br>(in/hr) | 1.39<br>8.6E-03<br>24.29<br>12.15 | 0.48<br>1.1E-03<br>3.02<br>1.51 | 1.43<br>9.6E-03<br>27.18<br>13.59 | 1.43<br>9.5E-03<br>26.8<br>13.4 | 1.7<br>1.8E-02<br>51.12<br>25.56 | 1.07<br>4.2E-03<br>11.84<br>5.92 |

Note:

[1] Soil Layers: SP-SM = Pooly-graded Sand with Silt
GW-GM = Well-graded Gravel with Silt and sand
SM = Poorly-graded Silty Sand

The design infiltration rate determined for each facility is factored down from the calculated hydraulic conductivity value to account for potential clogging, subsurface anisotropy that tends to reduce vertical hydraulic conductivity in natural alluvial and fluvio-glacial deposits, and possible limited maintenance over the long-term operational life of the facility. Other performance factors include siltation and bio-buildup that tend to reduce the effective infiltration rate over the long-term. This results in lower values being used for the infiltration facility design than are used for the groundwater mounding analysis.

The SEEP/W model uses the following values for saturated hydraulic conductivity:

- SM - silty fine to coarse sand with gravel: 7.8 x $10^{-3}$ centimeters per second (cm)/s) (22 ft/d)
- SP - fine to medium sand: 3.5 x $10^{-3}$ cm/s (10 ft/d)
- SP-SM – fine to coarse sand with silt and gravel 7.06 x $10^{-3}$ cm/s (20 ft/d)
- GP-GM – fine to coarse gravel with silt and sand 9.2 x $10^{-3}$ cm/s (26 ft/d)
- ML – silt with trace sand 7.06 x $10^{-6}$ cm/s (0.02 ft/d)

The silty fine to coarse sand with gravel and fine to coarse gravel with silt and sand layers will form the subgrade for the infiltration facilities. The assigned hydraulic conductivities are averages of the values indicated in Table 1 and conservative values obtained from 'Applied Hydrogeology' (Fetter 2001) to account for possible variation in soil permeability at different locations where subgrade soils have not been tested for sieve analysis .

SEEP/W is superior to other groundwater flow programs, such as MODFLOW or MODRET, for conducting a groundwater mounding analysis in that it explicitly models the unsaturated seepage occurring from the base of the infiltration facility to the water table and correctly represents the hydraulic properties of the soil layers present. It also includes a ponding function that will correctly account for water backing up above the infiltration elevation, should this occur.

Modeling of transient seepage through the vadose zone requires the specification of characteristic curves relating moisture content and hydraulic conductivity to the matric suction (sometimes known as soil water potential) for soils that are not fully saturated. In the absence of measured values for the site soils, published example curves from similar soils have been used as provided within the SEEP/W model.

## 3.0 MODEL SIMULATIONS

### 3.1. Initial Conditions

The SEEP/W model is first run to generate hydrogeologic conditions that would apply at the beginning of a peak storm, which coincides with the seasonal high groundwater elevation. This is accomplished by first running a steady-state simulation in SEEP/W, with no infiltration occurring, but with the boundary condition established to generate a water table at the seasonal high groundwater elevation of 64.76 feet. The results of this initial analysis are shown in Figure B, Steady State Conditions.

This initial step is important in establishing the correct pore pressure regime within the subsurface, including the layer of unsaturated soil above the water table just below the infiltration facility. The moisture content of this soil determines its effective hydraulic conductivity and, therefore, the initial infiltration rate during the early hours of the storm. These values are governed by the matric suction in the soil which varies with height above the water table. Soil water characteristic curves defining the soil properties of horizontal hydraulic conductivity and volumetric water content as functions of matric suction are specified within the SEEP/W model.

### 3.2. Infiltration Rate

The nominal design infiltration rate used by SCJ Alliance in their preliminary sizing and design of the stormwater infiltration facilities was 2 in/hr (NE facility) and 5.5 in/hr (SW facilities. To conservatively assess the worst-case degree of groundwater mounding that could be expected to occur, SEEP/W has been run to simulate the infiltration occurring during the 30-day peak runoff period for generation of stormwater from the Evergreen Ford Lincoln surface basin catchment draining to the proposed facilities.

Runoff hydrographs have been provided by SCJ Alliance for each of the proposed infiltration facilities, that indicate the amount of stormwater runoff developed hourly throughout the wettest water year in their continuous hydrologic simulation for the site, developed using the Western Washington Hydrology Model (WWHM) with the facilities sized for the design infiltration rates. For the groundwater mounding analysis, we focus on the wettest 30-day period of the simulation, which corresponds to the month of November 2006 in the WWHM time series dataset.

The runoff volumes are provided in cubic feet per second, entering the infiltration facility. The facility inflow rates are converted to equivalent infiltration rates assuming uniform distribution of inflowing stormwater at the base of the facility. The inflow volume is converted to cubic feet per hour, and then divided by the total basal area (in square feet) of the facility to give a resulting infiltration rate in feet per hour. This is then converted to feet per day for consistency with SEEP/W model units. The continuously varying infiltration rate is input to the SEEP/W model as a transient unit flux boundary condition (Figure C, No Mounding, 5 days and Figure D, Maximum Mounding, 9.25 days) applied at the floor of each infiltration trench (the boundary condition is represented by a series of downward pointing arrows in Figures C, D).

### 3.3. Results of the Mounding Analysis

The results of the groundwater mounding analysis are a set of predicted groundwater levels used to identify distinct periods of high stormwater infiltration and groundwater mounding during the WWHM time series simulation. The results range between periods of controlled runoff infiltration (Figure C), to the peak groundwater mounding conditions (Figure D). Peak mounding occurs around day nine of the November 2006 storm. The contours in these models depict equipotential lines of total hydraulic head, measured in feet and correspond to the elevation of mounding of the water table (blue dashed line).

The seepage velocities indicated by the SEEP/W model at the base of each facility provide a measure of the simulated infiltration rates achieved during facility operation. At peak mounding (Figure D) the seepage velocities are higher than the designed infiltration rates for the NE facility, 6.35 ft/d and lower than the designed infiltration rate for the SW facility, 5.02 ft/d, which is equivalent to peak infiltration rates of 3.175 in/hr (NE facility) and 2.51 in/hr (SW facility). At the SW facility, the reduced infiltration rate is likely caused by the back-up effect of groundwater mounding above the shallow water table, which reduces the hydraulic gradient at the base of the infiltration facility and lowers the effective infiltration rate that can be achieved in the facility.

### 3.4. Facility Resizing

The initial groundwater mounding results were discussed with SCJ Alliance and the facilities were resized based on lower design infiltration rates of 1.5 in/hr at the NE facility and 3 in/hr at the SW facility. The basal areas of the facilities were increased to 1,952 ft$^2$ and 9,860 ft$^2$, respectively, and updated inflow hydrographs from SCJ Alliance were recalculated to define new boundary conditions for each facility in the revised SEEP/W model. The revised model cross section showing the larger facilities and a finer model grid is presented in Figure E, SeepW Model Construction and Materials Updated Mesh Properties.

The results of the revised groundwater mounding analysis showed a significant reduction in the amount of groundwater mounding at 9.25 days into the simulation (Figure F, Resized Facility, 9.25 Days). The updated seepage velocities at 9.25 days are much less than the designed infiltration rates for both facilities, 0.529 ft/d (NE facility) and 0.564 ft/d (SW facility), which is equivalent to 0.265 in/hr and 0.282 in/hr, respectively. This represents a more normal condition for the very infrequent circumstances of a peak storm which would cause a short period when the groundwater mound would back up into the facility. The simulation shows that the water table would drop rapidly after the storm, allowing the treatment capacity of the biofilm zone at the base of the facility to be rapidly established for the protection of groundwater quality.

## 4.0 CONCLUSIONS

The relatively shallow depth to groundwater of 7 feet bgs at the project site constrains the vertical separation that can be achieved below the proposed infiltration facilities. Minimum vertical separation (without infiltration) under seasonal high groundwater conditions will be only 3 feet, and a mounding analysis must be accomplished to demonstrate that groundwater levels will not rise too high during major storm events and cause the stormwater infiltration facility to back-up and overflow.

Grain-size analysis of soil samples plus observations made during explorations conducted on site in October 2018 and presented in our Geotechnical Engineering Services Report, were used to develop a groundwater mounding analysis for two stormwater infiltration facilities being proposed for the site improvements. The mounding analysis was developed using SEEP/W, a finite-element groundwater flow and seepage modeling software package.

Inflow hydrographs developed by SCJ Alliance assuming preliminary design infiltration rates of 2 in/hr (NE facility) and 5.5 in/hr (SW facility) were used in the initial mounding analysis. The results indicated that groundwater levels will rise into and above the facilities, causing overflow during the peak storm period of the simulated runoff hydrographs used for the design of the facilities.

The initial groundwater mounding results were discussed with SCJ Alliance and the facilities were resized based on lower design infiltration rates of 1.5 in/hr at the NE facility and 3 in/hr at the SW facility and the basal areas of the facilities were increased. The revised groundwater mounding analysis with resized facilities using lower infiltration rates showed that the facilities will function as designed for most storm durations and intensities without overflowing.

Based on our analysis, this indicates that groundwater mounds will rise briefly into facilities under the highest intensity storms considered in the design time series. This is a normal operating condition for infiltration facilities which will operate for most storms with minimal mounding of the groundwater table, with unsaturated conditions at the base of the facilities to maintain treatment capacity in the subsurface soils.

## 5.0 LIMITATIONS

We have prepared this report for Evergreen Ford Lincoln, for Hydrogeologic Services, Groundwater Mounding Study. Evergreen Ford Lincoln may distribute copies of this report to authorized agents and regulatory agencies as may be required for the Project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of hydrogeology and stormwater infiltration in this area at the time this report was prepared. The conclusions, recommendations and opinions presented in this report are based on our professional knowledge, judgment and experience. No warranty, express or implied, applies to our services or this report.

Any electronic form, facsimile or hard copy of the original document (email, text, table and/or figure), if provided, and any attachments should be considered a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

## 6.0 REFERENCES

Devlin, J.F. 2015. HydrogeoSieveXL: an Excel-based tool to estimate hydraulic conductivity from grain-size analysis. Hydrogeology Journal Volume 23, Issue 4, pp 837-844

Fetter, C.W. 2001. Applied Hydrogeology (4th ed.), Prentice Hall, Upper Saddle River, NJ, 598p.

Sincerely,
GeoEngineers, Inc.

MICHAEL A.P. KENRICK
STATE OF WASHINGTON
31228
REGISTERED PROFESSIONAL ENGINEER

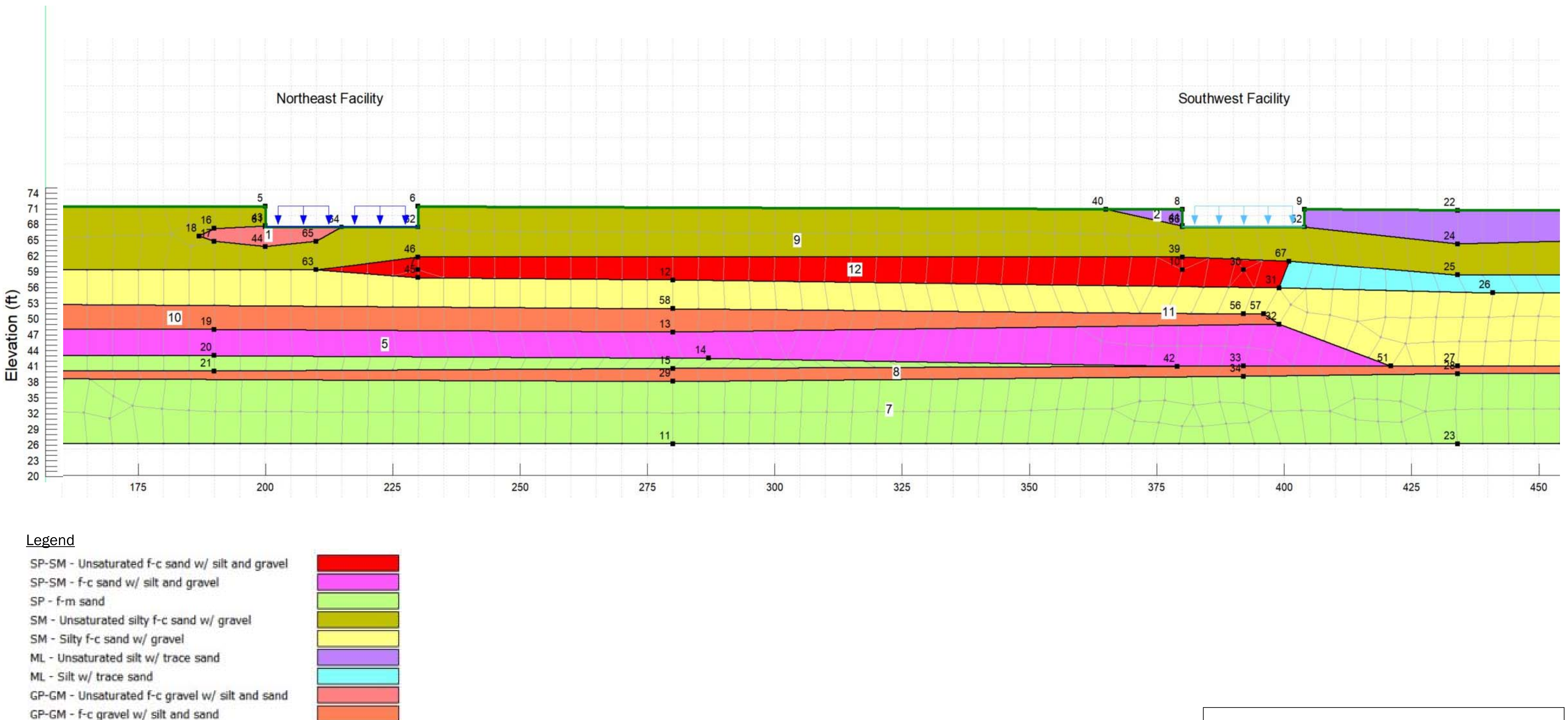
State of Washington
Hydrogeologist
1964
Licensed Geologist
Michael Kenrick

Wade Worthing
Wade C. Worthing
Staff Hydrogeologist

5/30/19
Michael A.P. Kenrick, PE, LHG
Senior Principal Hydrogeologist

WDW:MAPK:tlm

Attachments:
Figure A. SeepW Model Construction and Materials
Figure B. Steady State Conditions
Figure C. No Mounding, 5 Days
Figure D. Maximum Mounding, 9.25 Days
Figure E. SeepW Model Construction and Materials
Figure F. Former Maximum Mounding, 9.25 Days

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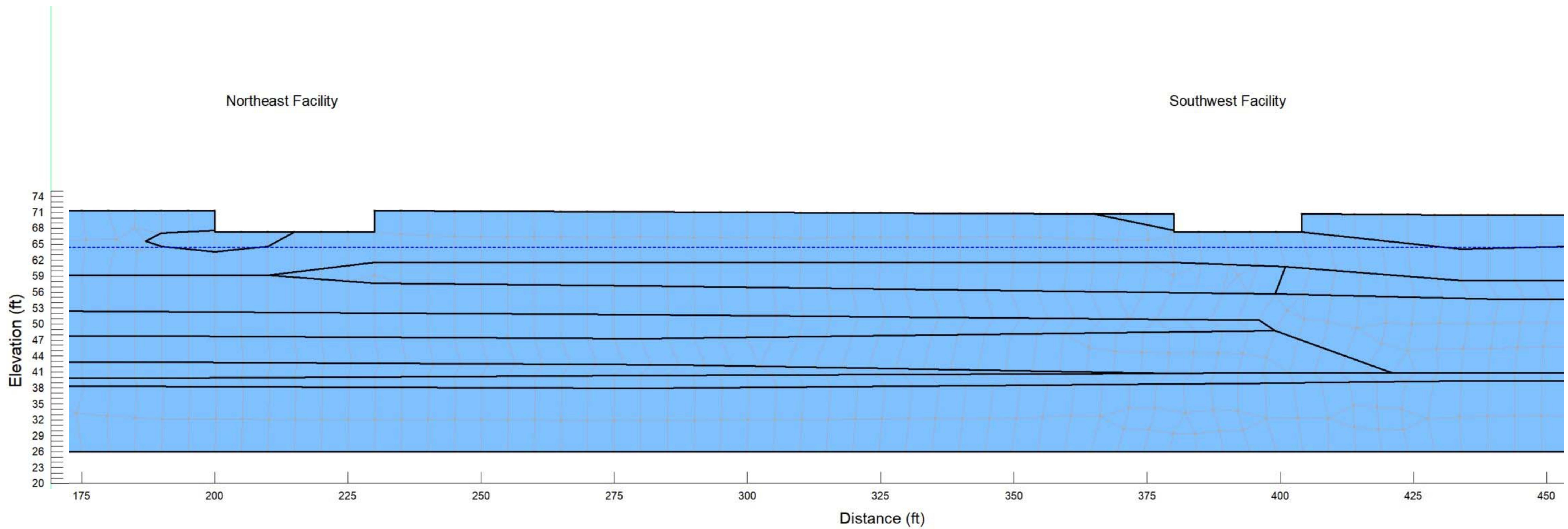
**SeepW Model Construction and Materials**

Evergreen Ford Lincoln
Issaquah, WA

GEOENGINEERS

**Figure A**

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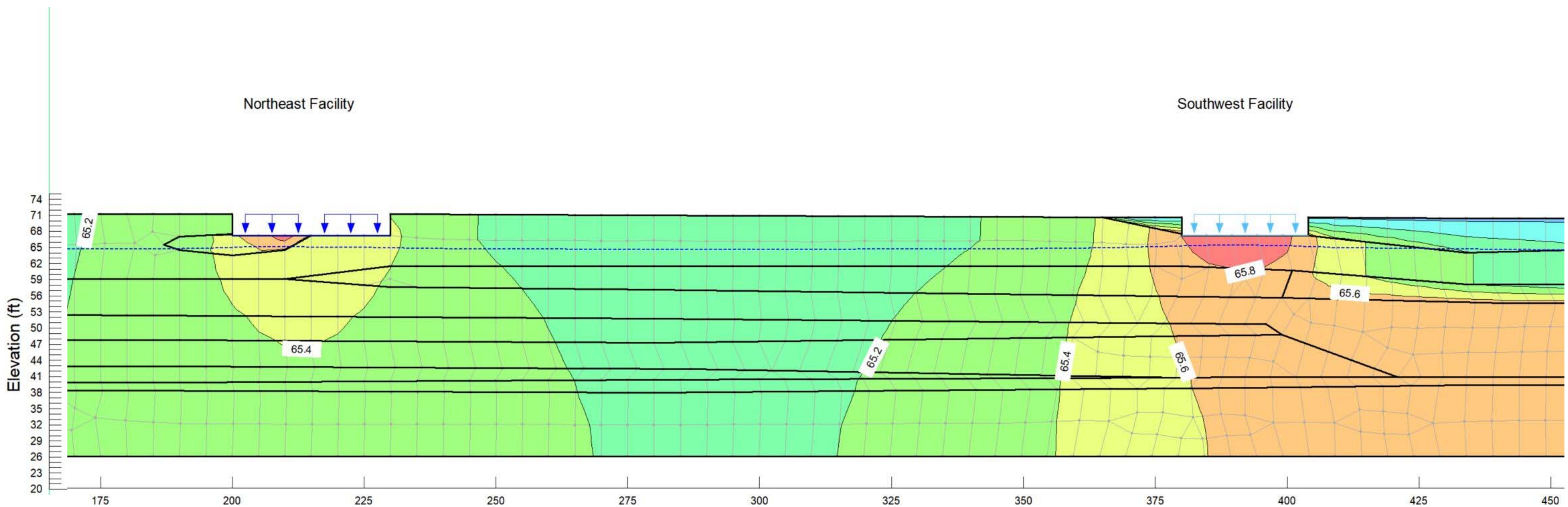
**Steady State Conditions**

Evergreen Ford Lincoln
Issaquah, WA

GeoEngineers

**Figure B**




**No Mounding, 5 Days**

Evergreen Ford Lincoln
Issaquah, WA

GeoEngineers

**Figure C**

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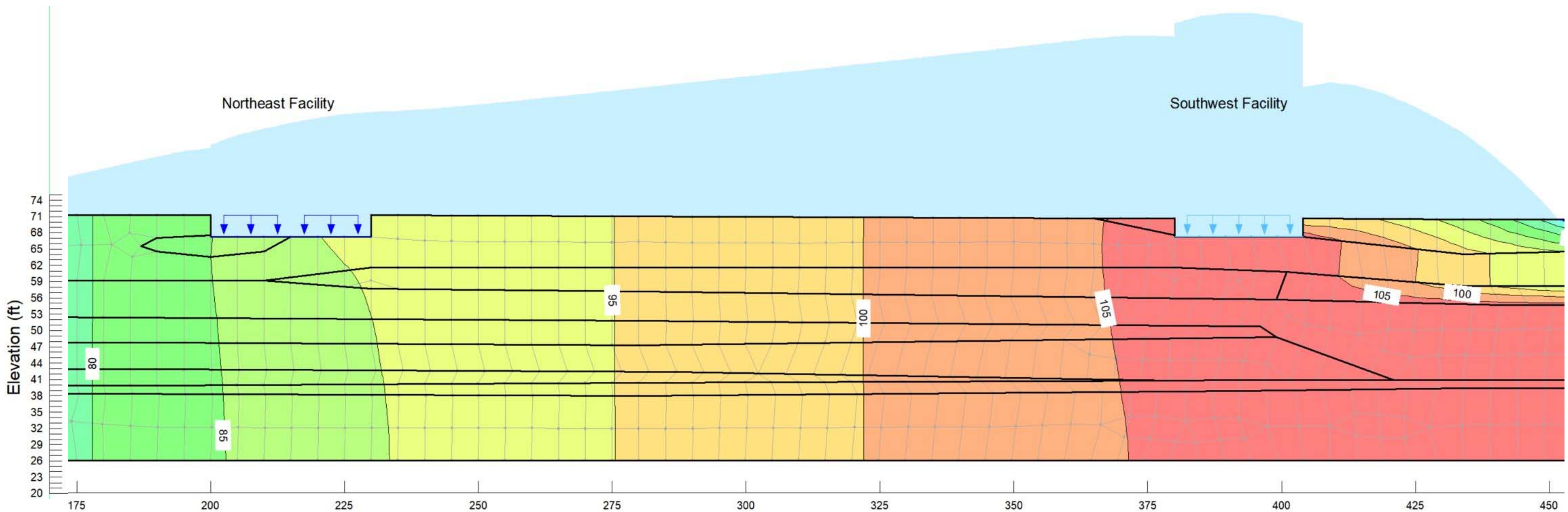


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Data Source:

**Maximum Mounding, 9.25 Days**

Evergreen Ford Lincoln
Issaquah, WA

GeoEngineers

**Figure D**

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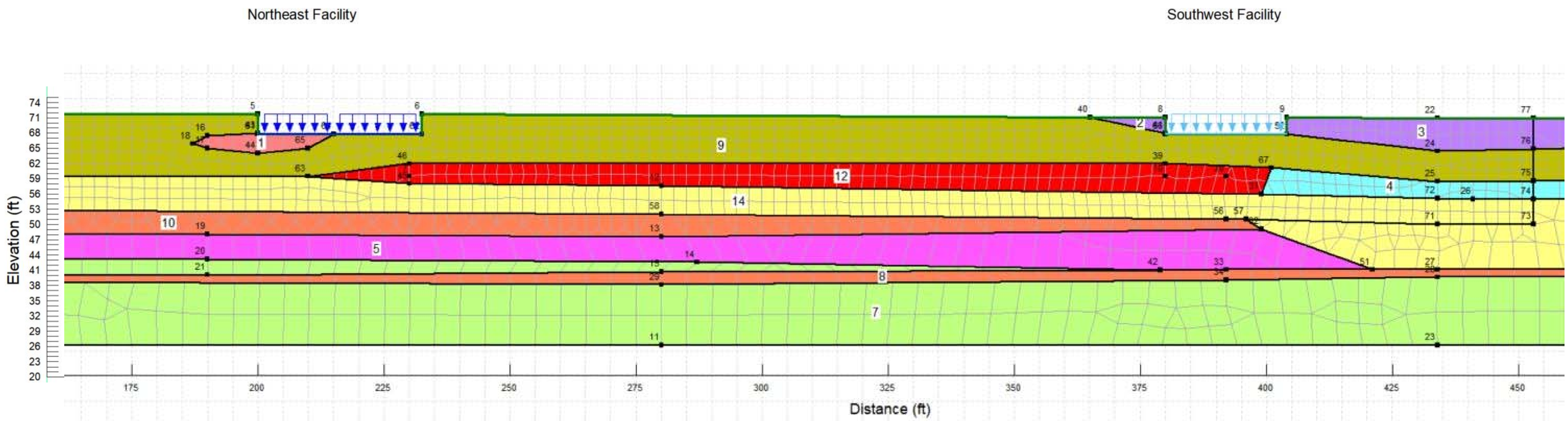


Legend

| | |
|---|---|
| SP-SM - Unsaturated f-c sand w/ silt and gravel | |
| SP-SM - f-c sand w/ silt and gravel | |
| SP - f-m sand | |
| SM - Unsaturated silty f-c sand w/ gravel | |
| SM - Silty f-c sand w/ gravel | |
| ML - Unsaturated silt w/ trace sand | |
| ML - Silt w/ trace sand | |
| GP-GM - Unsaturated f-c gravel w/ silt and sand | |
| GP-GM - f-c gravel w/ silt and sand | |

Notes:
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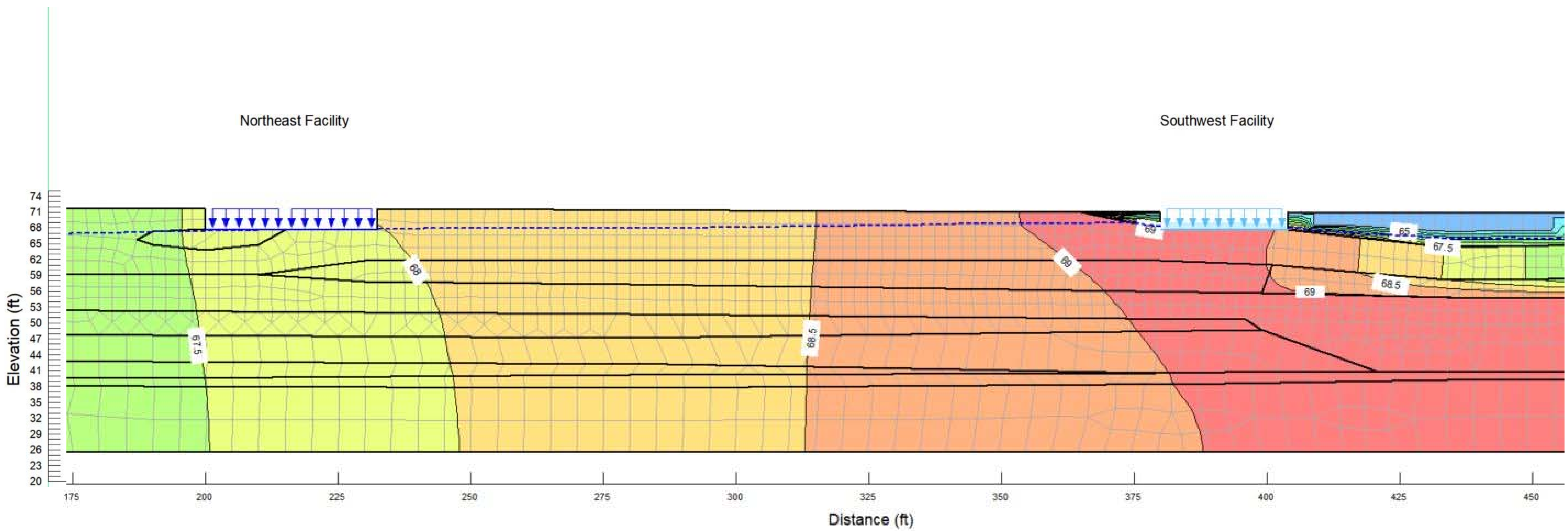
XXXXX-XXX-XX Date Exported: 04/09/15

**SeepW Model Construction and Materials**
**Updated Mesh Properties**

Evergreen Ford Lincoln
Issaquah, WA

GEOENGINEERS

**Figure E**




Notes:
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Data Source:

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**Resized Facility, 9.25 Days**
**Compare to Figure D**

Evergreen Ford Lincoln
Issaquah, WA

GeoEngineers

**Figure F**

# APPENDIX 6

## OPERATIONS AND MAINTENANCE MANUAL

**NOT INCLUDED FOR THIS SUBMITTAL**

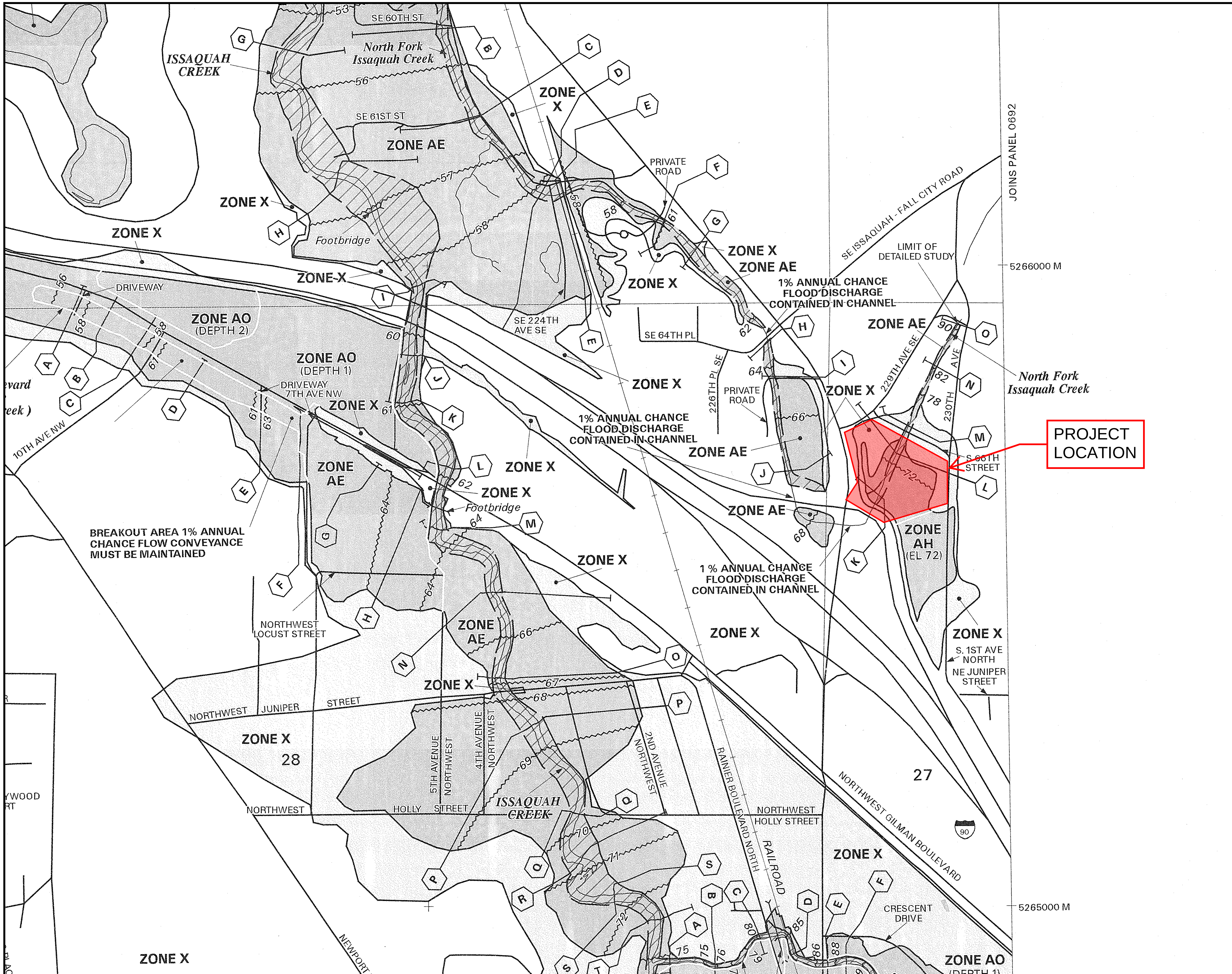
# APPENDIX 7

## CONSTRUCTION STORMWATER POLLUTION PREVENTION PLAN

**NOT INCLUDED FOR THIS SUBMITTAL**

# APPENDIX 8

## FEMA FLOOD INSURANCE MAP




MAP SCALE 1" = 500'

500 1000 FEET

METERS

PANEL 0691H

# FIRM
## FLOOD INSURANCE RATE MAP
## KING COUNTY, WASHINGTON AND INCORPORATED AREAS

## PANEL 691 OF 1725

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

| COMMUNITY | NUMBER | PANEL | SUFFIX |
|---|---|---|---|
| KING COUNTY, UNINCORPORATED AREAS | 530071 | 0691 | H |
| ISSAQUAH, CITY OF | 530079 | 0691 | H |

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

**MAP NUMBER 53033C0691H**

**MAP REVISED: APRIL 19, 2005**

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

# APPENDIX 9

## DESIGN CALCULATIONS AND COMPUTATIONS

# WWHM2012

# PROJECT REPORT

## BASIN 1:
## FLOW CONTROL

# General Model Information

| | |
|---|---|
| Project Name: | 1883.01 Issaquah Evergreen Ford Vault |
| Site Name: | |
| Site Address: | |
| City: | |
| Report Date: | 6/11/2019 |
| Gage: | Seatac |
| Data Start: | 1948/10/01 |
| Data End: | 2009/09/30 |
| Timestep: | 15 Minute |
| Precip Scale: | 1.333 |
| Version Date: | 2018/10/10 |
| Version: | 4.2.16 |

## POC Thresholds

| | |
|---|---|
| Low Flow Threshold for POC1: | 50 Percent of the 2 Year |
| High Flow Threshold for POC1: | 50 Year |

# Landuse Basin Data

## *Predeveloped Land Use*

### Basin 1

| | |
|---|---|
| Bypass: | No |
| GroundWater: | No |
| Pervious Land Use | acre |
| C, Forest, Flat | 3.1 |
| Pervious Total | 3.1 |
| Impervious Land Use | acre |
| Impervious Total | 0 |
| Basin Total | 3.1 |

Element Flows To:

| Surface | Interflow | Groundwater |
|---|---|---|

*Mitigated Land Use*

## Basin 1

| | |
|---|---|
| Bypass: | No |
| GroundWater: | No |
| Pervious Land Use | acre |
| C, Lawn, Flat | 0.25 |
| Pervious Total | 0.25 |
| Impervious Land Use | acre |
| ROOF TOPS FLAT | 1 |
| SIDEWALKS FLAT | 0.15 |
| PARKING FLAT | 1.7 |
| Impervious Total | 2.85 |
| Basin Total | 3.1 |

Element Flows To:

| Surface | Interflow | Groundwater |
|---|---|---|
| Vault | Vault | |

## *Mitigated Routing*

### Vault

| | | |
|---|---|---|
| Width: | 102 ft. | |
| Length: | 102 ft. | |
| Depth: | 8 ft. | |
| Discharge Structure | | |
| Riser Height: | 7 ft. | |
| Riser Diameter: | 18 in. | |
| Orifice 1 Diameter: | 1.28 in. | Elevation:0 ft. |
| Orifice 2 Diameter: | 2.34 in. | Elevation:5.6 ft. |
| Orifice 3 Diameter: | 1.48 in. | Elevation:6.65 ft. |
| Element Flows To: | | |
| Outlet 1 | Outlet 2 | |

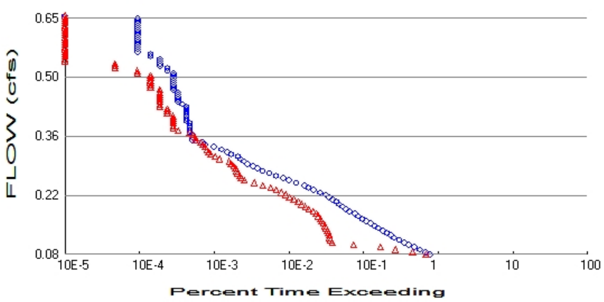
Vault Hydraulic Table

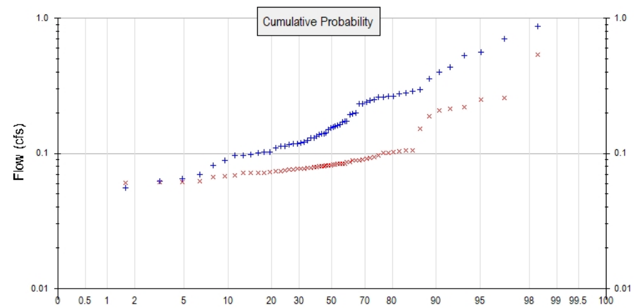
| **Stage(feet)** | **Area(ac.)** | **Volume(ac-ft.)** | **Discharge(cfs)** | **Infilt(cfs)** |
|---|---|---|---|---|
| 0.0000 | 0.238 | 0.000 | 0.000 | 0.000 |
| 0.0889 | 0.238 | 0.021 | 0.013 | 0.000 |
| 0.1778 | 0.238 | 0.042 | 0.018 | 0.000 |
| 0.2667 | 0.238 | 0.063 | 0.023 | 0.000 |
| 0.3556 | 0.238 | 0.084 | 0.026 | 0.000 |
| 0.4444 | 0.238 | 0.106 | 0.029 | 0.000 |
| 0.5333 | 0.238 | 0.127 | 0.032 | 0.000 |
| 0.6222 | 0.238 | 0.148 | 0.035 | 0.000 |
| 0.7111 | 0.238 | 0.169 | 0.037 | 0.000 |
| 0.8000 | 0.238 | 0.191 | 0.039 | 0.000 |
| 0.8889 | 0.238 | 0.212 | 0.041 | 0.000 |
| 0.9778 | 0.238 | 0.233 | 0.044 | 0.000 |
| 1.0667 | 0.238 | 0.254 | 0.045 | 0.000 |
| 1.1556 | 0.238 | 0.276 | 0.047 | 0.000 |
| 1.2444 | 0.238 | 0.297 | 0.049 | 0.000 |
| 1.3333 | 0.238 | 0.318 | 0.051 | 0.000 |
| 1.4222 | 0.238 | 0.339 | 0.053 | 0.000 |
| 1.5111 | 0.238 | 0.360 | 0.054 | 0.000 |
| 1.6000 | 0.238 | 0.382 | 0.056 | 0.000 |
| 1.6889 | 0.238 | 0.403 | 0.057 | 0.000 |
| 1.7778 | 0.238 | 0.424 | 0.059 | 0.000 |
| 1.8667 | 0.238 | 0.445 | 0.060 | 0.000 |
| 1.9556 | 0.238 | 0.467 | 0.062 | 0.000 |
| 2.0444 | 0.238 | 0.488 | 0.063 | 0.000 |
| 2.1333 | 0.238 | 0.509 | 0.064 | 0.000 |
| 2.2222 | 0.238 | 0.530 | 0.066 | 0.000 |
| 2.3111 | 0.238 | 0.552 | 0.067 | 0.000 |
| 2.4000 | 0.238 | 0.573 | 0.068 | 0.000 |
| 2.4889 | 0.238 | 0.594 | 0.070 | 0.000 |
| 2.5778 | 0.238 | 0.615 | 0.071 | 0.000 |
| 2.6667 | 0.238 | 0.636 | 0.072 | 0.000 |
| 2.7556 | 0.238 | 0.658 | 0.073 | 0.000 |
| 2.8444 | 0.238 | 0.679 | 0.075 | 0.000 |
| 2.9333 | 0.238 | 0.700 | 0.076 | 0.000 |
| 3.0222 | 0.238 | 0.721 | 0.077 | 0.000 |
| 3.1111 | 0.238 | 0.743 | 0.078 | 0.000 |
| 3.2000 | 0.238 | 0.764 | 0.079 | 0.000 |
| 3.2889 | 0.238 | 0.785 | 0.080 | 0.000 |

| | | | | |
|---|---|---|---|---|
| 3.3778 | 0.238 | 0.806 | 0.081 | 0.000 |
| 3.4667 | 0.238 | 0.828 | 0.082 | 0.000 |
| 3.5556 | 0.238 | 0.849 | 0.083 | 0.000 |
| 3.6444 | 0.238 | 0.870 | 0.084 | 0.000 |
| 3.7333 | 0.238 | 0.891 | 0.085 | 0.000 |
| 3.8222 | 0.238 | 0.912 | 0.086 | 0.000 |
| 3.9111 | 0.238 | 0.934 | 0.087 | 0.000 |
| 4.0000 | 0.238 | 0.955 | 0.088 | 0.000 |
| 4.0889 | 0.238 | 0.976 | 0.089 | 0.000 |
| 4.1778 | 0.238 | 0.997 | 0.090 | 0.000 |
| 4.2667 | 0.238 | 1.019 | 0.091 | 0.000 |
| 4.3556 | 0.238 | 1.040 | 0.092 | 0.000 |
| 4.4444 | 0.238 | 1.061 | 0.093 | 0.000 |
| 4.5333 | 0.238 | 1.082 | 0.094 | 0.000 |
| 4.6222 | 0.238 | 1.104 | 0.095 | 0.000 |
| 4.7111 | 0.238 | 1.125 | 0.096 | 0.000 |
| 4.8000 | 0.238 | 1.146 | 0.097 | 0.000 |
| 4.8889 | 0.238 | 1.167 | 0.098 | 0.000 |
| 4.9778 | 0.238 | 1.188 | 0.099 | 0.000 |
| 5.0667 | 0.238 | 1.210 | 0.100 | 0.000 |
| 5.1556 | 0.238 | 1.231 | 0.101 | 0.000 |
| 5.2444 | 0.238 | 1.252 | 0.101 | 0.000 |
| 5.3333 | 0.238 | 1.273 | 0.102 | 0.000 |
| 5.4222 | 0.238 | 1.295 | 0.103 | 0.000 |
| 5.5111 | 0.238 | 1.316 | 0.104 | 0.000 |
| 5.6000 | 0.238 | 1.337 | 0.105 | 0.000 |
| 5.6889 | 0.238 | 1.358 | 0.150 | 0.000 |
| 5.7778 | 0.238 | 1.380 | 0.169 | 0.000 |
| 5.8667 | 0.238 | 1.401 | 0.184 | 0.000 |
| 5.9556 | 0.238 | 1.422 | 0.197 | 0.000 |
| 6.0444 | 0.238 | 1.443 | 0.208 | 0.000 |
| 6.1333 | 0.238 | 1.464 | 0.218 | 0.000 |
| 6.2222 | 0.238 | 1.486 | 0.228 | 0.000 |
| 6.3111 | 0.238 | 1.507 | 0.237 | 0.000 |
| 6.4000 | 0.238 | 1.528 | 0.245 | 0.000 |
| 6.4889 | 0.238 | 1.549 | 0.253 | 0.000 |
| 6.5778 | 0.238 | 1.571 | 0.261 | 0.000 |
| 6.6667 | 0.238 | 1.592 | 0.275 | 0.000 |
| 6.7556 | 0.238 | 1.613 | 0.294 | 0.000 |
| 6.8444 | 0.238 | 1.634 | 0.308 | 0.000 |
| 6.9333 | 0.238 | 1.656 | 0.320 | 0.000 |
| 7.0222 | 0.238 | 1.677 | 0.384 | 0.000 |
| 7.1111 | 0.238 | 1.698 | 0.929 | 0.000 |
| 7.2000 | 0.238 | 1.719 | 1.755 | 0.000 |
| 7.2889 | 0.238 | 1.740 | 2.735 | 0.000 |
| 7.3778 | 0.238 | 1.762 | 3.755 | 0.000 |
| 7.4667 | 0.238 | 1.783 | 4.704 | 0.000 |
| 7.5556 | 0.238 | 1.804 | 5.483 | 0.000 |
| 7.6444 | 0.238 | 1.825 | 6.044 | 0.000 |
| 7.7333 | 0.238 | 1.847 | 6.416 | 0.000 |
| 7.8222 | 0.238 | 1.868 | 6.836 | 0.000 |
| 7.9111 | 0.238 | 1.889 | 7.182 | 0.000 |
| 8.0000 | 0.238 | 1.910 | 7.511 | 0.000 |
| 8.0889 | 0.238 | 1.932 | 7.827 | 0.000 |
| 8.1778 | 0.000 | 0.000 | 8.130 | 0.000 |

# Analysis Results

## POC 1




+ Predeveloped x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 3.1
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 0.25
Total Impervious Area: 2.85

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

| **Return Period** | **Flow(cfs)** |
|---|---|
| 2 year | 0.160526 |
| 5 year | 0.274848 |
| 10 year | 0.370591 |
| 25 year | 0.516745 |
| 50 year | 0.645445 |
| 100 year | 0.792464 |

Flow Frequency Return Periods for Mitigated. POC #1

| **Return Period** | **Flow(cfs)** |
|---|---|
| 2 year | 0.087459 |
| 5 year | 0.127317 |
| 10 year | 0.160112 |
| 25 year | 0.209834 |
| 50 year | 0.253584 |
| 100 year | 0.303712 |

## Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

| **Year** | **Predeveloped** | **Mitigated** |
|---|---|---|
| 1949 | 0.233 | 0.068 |
| 1950 | 0.261 | 0.084 |
| 1951 | 0.297 | 0.209 |
| 1952 | 0.102 | 0.061 |
| 1953 | 0.088 | 0.077 |
| 1954 | 0.125 | 0.079 |
| 1955 | 0.193 | 0.080 |
| 1956 | 0.172 | 0.090 |
| 1957 | 0.158 | 0.078 |
| 1958 | 0.142 | 0.083 |

| | | |
|---|---|---|
| 1959 | 0.118 | 0.074 |
| 1960 | 0.259 | 0.152 |
| 1961 | 0.120 | 0.086 |
| 1962 | 0.082 | 0.061 |
| 1963 | 0.121 | 0.079 |
| 1964 | 0.157 | 0.082 |
| 1965 | 0.131 | 0.091 |
| 1966 | 0.097 | 0.075 |
| 1967 | 0.265 | 0.084 |
| 1968 | 0.139 | 0.076 |
| 1969 | 0.138 | 0.072 |
| 1970 | 0.117 | 0.075 |
| 1971 | 0.161 | 0.083 |
| 1972 | 0.233 | 0.101 |
| 1973 | 0.113 | 0.089 |
| 1974 | 0.150 | 0.082 |
| 1975 | 0.196 | 0.078 |
| 1976 | 0.139 | 0.080 |
| 1977 | 0.096 | 0.072 |
| 1978 | 0.112 | 0.083 |
| 1979 | 0.070 | 0.061 |
| 1980 | 0.434 | 0.189 |
| 1981 | 0.098 | 0.076 |
| 1982 | 0.277 | 0.100 |
| 1983 | 0.162 | 0.081 |
| 1984 | 0.101 | 0.067 |
| 1985 | 0.062 | 0.074 |
| 1986 | 0.247 | 0.093 |
| 1987 | 0.238 | 0.103 |
| 1988 | 0.103 | 0.072 |
| 1989 | 0.065 | 0.071 |
| 1990 | 0.874 | 0.105 |
| 1991 | 0.354 | 0.104 |
| 1992 | 0.130 | 0.083 |
| 1993 | 0.117 | 0.068 |
| 1994 | 0.056 | 0.060 |
| 1995 | 0.155 | 0.086 |
| 1996 | 0.398 | 0.213 |
| 1997 | 0.289 | 0.256 |
| 1998 | 0.134 | 0.071 |
| 1999 | 0.530 | 0.105 |
| 2000 | 0.109 | 0.089 |
| 2001 | 0.036 | 0.062 |
| 2002 | 0.200 | 0.096 |
| 2003 | 0.265 | 0.077 |
| 2004 | 0.280 | 0.248 |
| 2005 | 0.173 | 0.081 |
| 2006 | 0.171 | 0.089 |
| 2007 | 0.700 | 0.539 |
| 2008 | 0.564 | 0.221 |
| 2009 | 0.249 | 0.094 |

## Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

| Rank | Predeveloped | Mitigated |
|---|---|---|
| 1 | 0.8744 | 0.5392 |
| 2 | 0.6999 | 0.2562 |
| 3 | 0.5643 | 0.2482 |

| | | |
|---|---|---|
| 4 | 0.5295 | 0.2213 |
| 5 | 0.4342 | 0.2125 |
| 6 | 0.3985 | 0.2087 |
| 7 | 0.3540 | 0.1890 |
| 8 | 0.2972 | 0.1521 |
| 9 | 0.2886 | 0.1051 |
| 10 | 0.2802 | 0.1050 |
| 11 | 0.2767 | 0.1036 |
| 12 | 0.2652 | 0.1027 |
| 13 | 0.2649 | 0.1005 |
| 14 | 0.2612 | 0.1004 |
| 15 | 0.2589 | 0.0964 |
| 16 | 0.2488 | 0.0942 |
| 17 | 0.2466 | 0.0930 |
| 18 | 0.2381 | 0.0915 |
| 19 | 0.2329 | 0.0897 |
| 20 | 0.2328 | 0.0891 |
| 21 | 0.1998 | 0.0890 |
| 22 | 0.1958 | 0.0888 |
| 23 | 0.1926 | 0.0864 |
| 24 | 0.1733 | 0.0863 |
| 25 | 0.1716 | 0.0841 |
| 26 | 0.1710 | 0.0838 |
| 27 | 0.1622 | 0.0834 |
| 28 | 0.1606 | 0.0832 |
| 29 | 0.1581 | 0.0828 |
| 30 | 0.1568 | 0.0826 |
| 31 | 0.1552 | 0.0817 |
| 32 | 0.1496 | 0.0817 |
| 33 | 0.1422 | 0.0813 |
| 34 | 0.1391 | 0.0806 |
| 35 | 0.1391 | 0.0804 |
| 36 | 0.1380 | 0.0797 |
| 37 | 0.1340 | 0.0794 |
| 38 | 0.1307 | 0.0791 |
| 39 | 0.1301 | 0.0781 |
| 40 | 0.1247 | 0.0775 |
| 41 | 0.1209 | 0.0768 |
| 42 | 0.1195 | 0.0767 |
| 43 | 0.1180 | 0.0764 |
| 44 | 0.1171 | 0.0759 |
| 45 | 0.1169 | 0.0753 |
| 46 | 0.1130 | 0.0748 |
| 47 | 0.1123 | 0.0736 |
| 48 | 0.1094 | 0.0735 |
| 49 | 0.1026 | 0.0724 |
| 50 | 0.1020 | 0.0719 |
| 51 | 0.1013 | 0.0715 |
| 52 | 0.0983 | 0.0714 |
| 53 | 0.0965 | 0.0713 |
| 54 | 0.0961 | 0.0685 |
| 55 | 0.0885 | 0.0676 |
| 56 | 0.0818 | 0.0669 |
| 57 | 0.0700 | 0.0620 |
| 58 | 0.0648 | 0.0615 |
| 59 | 0.0624 | 0.0611 |
| 60 | 0.0557 | 0.0609 |
| 61 | 0.0363 | 0.0602 |

## Duration Flows

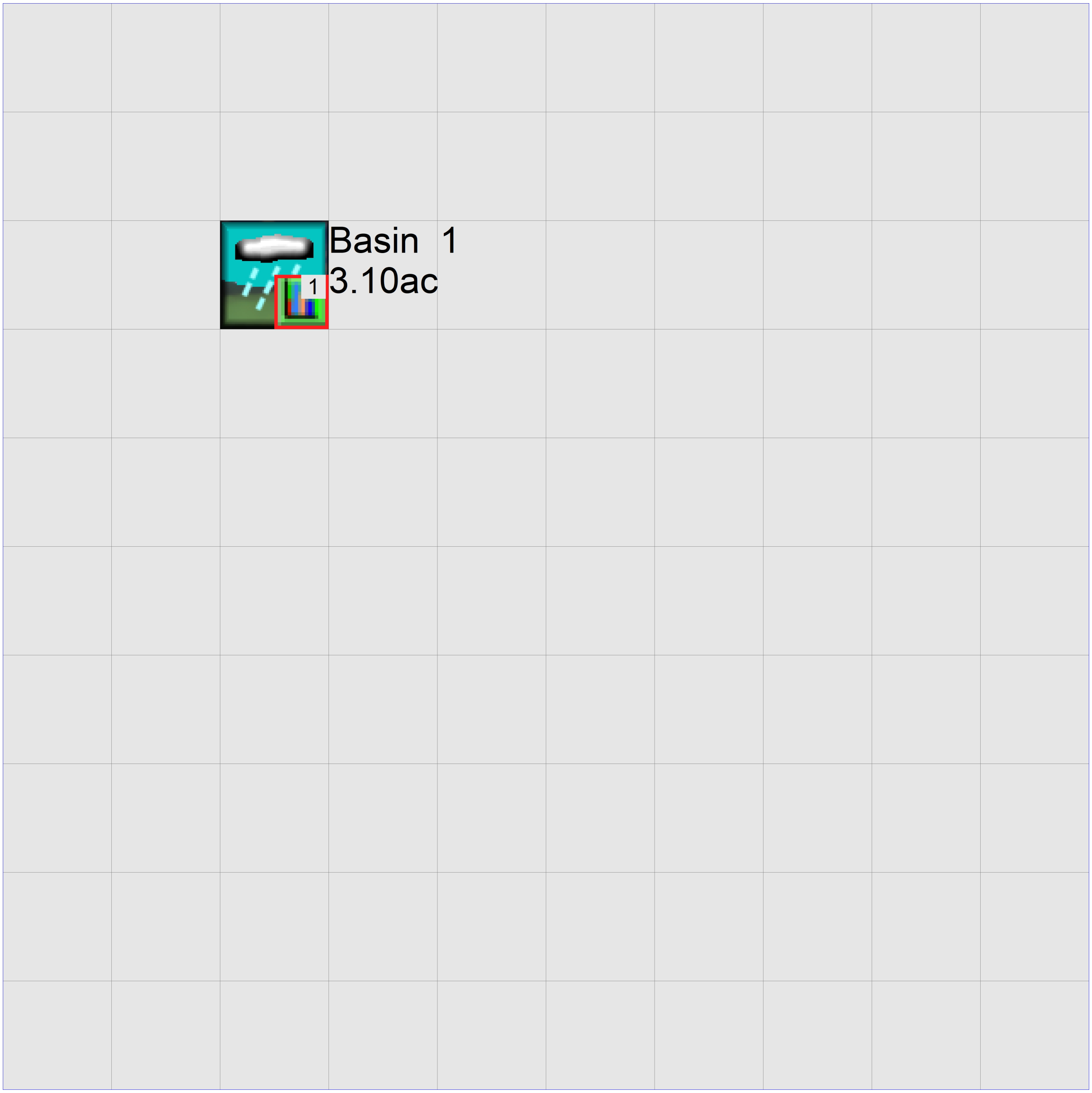
The Facility PASSED

| Flow(cfs) | Predev | Mit | Percentage | Pass/Fail |
|---|---|---|---|---|
| 0.0803 | 16495 | 14600 | 88 | Pass |
| 0.0860 | 14292 | 9629 | 67 | Pass |
| 0.0917 | 11993 | 5713 | 47 | Pass |
| 0.0974 | 10087 | 3598 | 35 | Pass |
| 0.1031 | 8491 | 1569 | 18 | Pass |
| 0.1088 | 7490 | 811 | 10 | Pass |
| 0.1145 | 6389 | 780 | 12 | Pass |
| 0.1202 | 5480 | 754 | 13 | Pass |
| 0.1259 | 4928 | 739 | 14 | Pass |
| 0.1316 | 4344 | 720 | 16 | Pass |
| 0.1374 | 3829 | 701 | 18 | Pass |
| 0.1431 | 3343 | 677 | 20 | Pass |
| 0.1488 | 2988 | 653 | 21 | Pass |
| 0.1545 | 2618 | 615 | 23 | Pass |
| 0.1602 | 2293 | 585 | 25 | Pass |
| 0.1659 | 2051 | 560 | 27 | Pass |
| 0.1716 | 1824 | 531 | 29 | Pass |
| 0.1773 | 1619 | 498 | 30 | Pass |
| 0.1830 | 1380 | 450 | 32 | Pass |
| 0.1887 | 1235 | 393 | 31 | Pass |
| 0.1944 | 1116 | 354 | 31 | Pass |
| 0.2002 | 1005 | 316 | 31 | Pass |
| 0.2059 | 922 | 286 | 31 | Pass |
| 0.2116 | 825 | 247 | 29 | Pass |
| 0.2173 | 734 | 216 | 29 | Pass |
| 0.2230 | 672 | 192 | 28 | Pass |
| 0.2287 | 562 | 169 | 30 | Pass |
| 0.2344 | 456 | 144 | 31 | Pass |
| 0.2401 | 389 | 121 | 31 | Pass |
| 0.2458 | 342 | 96 | 28 | Pass |
| 0.2515 | 266 | 73 | 27 | Pass |
| 0.2572 | 215 | 54 | 25 | Pass |
| 0.2629 | 180 | 49 | 27 | Pass |
| 0.2687 | 143 | 47 | 32 | Pass |
| 0.2744 | 116 | 44 | 37 | Pass |
| 0.2801 | 94 | 42 | 44 | Pass |
| 0.2858 | 80 | 41 | 51 | Pass |
| 0.2915 | 70 | 39 | 55 | Pass |
| 0.2972 | 59 | 36 | 61 | Pass |
| 0.3029 | 53 | 34 | 64 | Pass |
| 0.3086 | 48 | 25 | 52 | Pass |
| 0.3143 | 44 | 22 | 50 | Pass |
| 0.3200 | 36 | 19 | 52 | Pass |
| 0.3257 | 32 | 18 | 56 | Pass |
| 0.3315 | 27 | 17 | 62 | Pass |
| 0.3372 | 21 | 16 | 76 | Pass |
| 0.3429 | 16 | 15 | 93 | Pass |
| 0.3486 | 15 | 14 | 93 | Pass |
| 0.3543 | 11 | 12 | 109 | Pass |
| 0.3600 | 11 | 12 | 109 | Pass |
| 0.3657 | 11 | 11 | 100 | Pass |
| 0.3714 | 10 | 10 | 100 | Pass |
| 0.3771 | 10 | 7 | 70 | Pass |

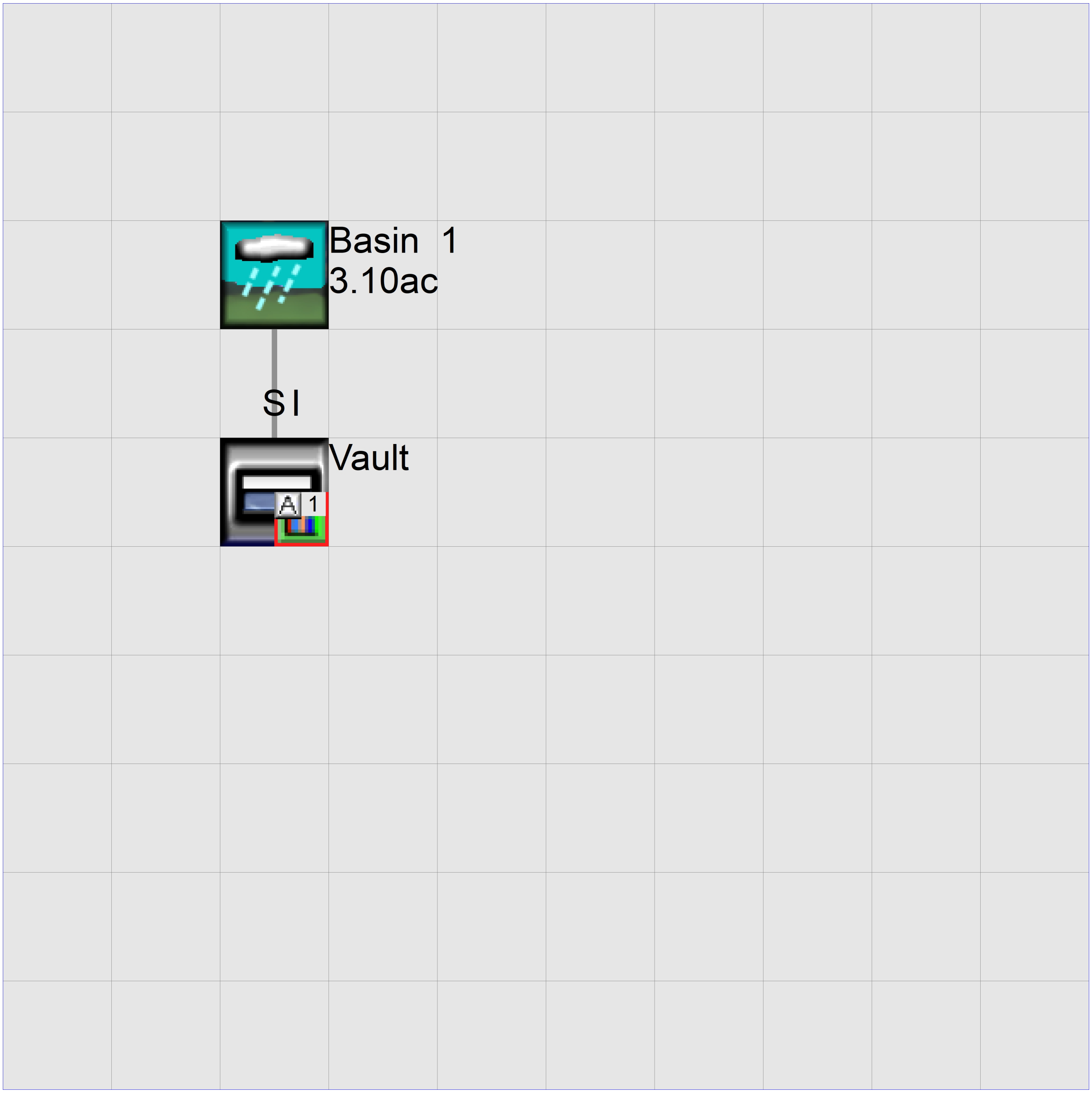
| | | | | |
|---|---|---|---|---|
| 0.3828 | 10 | 6 | 60 | Pass |
| 0.3885 | 10 | 6 | 60 | Pass |
| 0.3943 | 10 | 6 | 60 | Pass |
| 0.4000 | 9 | 6 | 66 | Pass |
| 0.4057 | 9 | 6 | 66 | Pass |
| 0.4114 | 9 | 6 | 66 | Pass |
| 0.4171 | 9 | 5 | 55 | Pass |
| 0.4228 | 9 | 5 | 55 | Pass |
| 0.4285 | 9 | 5 | 55 | Pass |
| 0.4342 | 9 | 4 | 44 | Pass |
| 0.4399 | 8 | 4 | 50 | Pass |
| 0.4456 | 7 | 4 | 57 | Pass |
| 0.4513 | 7 | 4 | 57 | Pass |
| 0.4571 | 7 | 4 | 57 | Pass |
| 0.4628 | 7 | 4 | 57 | Pass |
| 0.4685 | 7 | 4 | 57 | Pass |
| 0.4742 | 6 | 4 | 66 | Pass |
| 0.4799 | 6 | 3 | 50 | Pass |
| 0.4856 | 6 | 3 | 50 | Pass |
| 0.4913 | 6 | 3 | 50 | Pass |
| 0.4970 | 6 | 3 | 50 | Pass |
| 0.5027 | 6 | 3 | 50 | Pass |
| 0.5084 | 6 | 3 | 50 | Pass |
| 0.5141 | 6 | 2 | 33 | Pass |
| 0.5198 | 5 | 2 | 40 | Pass |
| 0.5256 | 5 | 1 | 20 | Pass |
| 0.5313 | 4 | 1 | 25 | Pass |
| 0.5370 | 4 | 1 | 25 | Pass |
| 0.5427 | 4 | 0 | 0 | Pass |
| 0.5484 | 4 | 0 | 0 | Pass |
| 0.5541 | 3 | 0 | 0 | Pass |
| 0.5598 | 3 | 0 | 0 | Pass |
| 0.5655 | 2 | 0 | 0 | Pass |
| 0.5712 | 2 | 0 | 0 | Pass |
| 0.5769 | 2 | 0 | 0 | Pass |
| 0.5826 | 2 | 0 | 0 | Pass |
| 0.5884 | 2 | 0 | 0 | Pass |
| 0.5941 | 2 | 0 | 0 | Pass |
| 0.5998 | 2 | 0 | 0 | Pass |
| 0.6055 | 2 | 0 | 0 | Pass |
| 0.6112 | 2 | 0 | 0 | Pass |
| 0.6169 | 2 | 0 | 0 | Pass |
| 0.6226 | 2 | 0 | 0 | Pass |
| 0.6283 | 2 | 0 | 0 | Pass |
| 0.6340 | 2 | 0 | 0 | Pass |
| 0.6397 | 2 | 0 | 0 | Pass |
| 0.6454 | 2 | 0 | 0 | Pass |

# Appendix

## Predeveloped Schematic



*Mitigated Schematic*



# Disclaimer

## Legal Notice



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6200 Capitol Blvd. Ste F
Olympia, WA. 98501
Toll Free 1(866)943-0304
Local (360)943-0304

www.clearcreeksolutions.com

# WWHM2012

# PROJECT REPORT

## BASIN 2: FLOW CONTROL
## BASIN 3: TREATMENT

# *General Model Information*

| | |
|---|---|
| Project Name: | 1883.01 Issaquah Evergreen Ford Community Space |
| Site Name: | |
| Site Address: | |
| City: | |
| Report Date: | 4/23/2019 |
| Gage: | Seatac |
| Data Start: | 1948/10/01 |
| Data End: | 2009/09/30 |
| Timestep: | 15 Minute |
| Precip Scale: | 1.333 |
| Version Date: | 2018/10/10 |
| Version: | 4.2.16 |

## *POC Thresholds*

| | |
|---|---|
| Low Flow Threshold for POC1: | 50 Percent of the 2 Year |
| High Flow Threshold for POC1: | 50 Year |

# *Landuse Basin Data*
## *Predeveloped Land Use*

### Basin 1

| | |
|---|---|
| Bypass: | No |
| GroundWater: | No |
| Pervious Land Use | acre |
| A B, Forest, Flat | 0.34 |
| Pervious Total | 0.34 |
| Impervious Land Use | acre |
| Impervious Total | 0 |
| Basin Total | 0.34 |

Element Flows To:

| Surface | Interflow | Groundwater |
|---|---|---|

## *Mitigated Land Use*

### Basin 1

| | |
|---|---|
| Bypass: | No |
| GroundWater: | No |
| Pervious Land Use | acre |
| A B, Lawn, Flat | 0.07 |
| Pervious Total | 0.07 |
| Impervious Land Use | acre |
| SIDEWALKS FLAT | 0.09 |
| PARKING FLAT | 0.18 |
| Impervious Total | 0.27 |
| Basin Total | 0.34 |

Element Flows To:

| Surface | Interflow | Groundwater |
|---|---|---|
| Gravel Trench Bed 1 | Gravel Trench Bed 1 | |

*Mitigated Routing*

## Gravel Trench Bed 1

| | |
|---|---|
| Bottom Length: | 100.00 ft. |
| Bottom Width: | 10.00 ft. |
| Trench bottom slope 1: | 3 To 1 |
| Trench Left side slope 0: | 3 To 1 |
| Trench right side slope 2: | 3 To 1 |
| Material thickness of first layer: | 1.5 |
| Pour Space of material for first layer: | 0.4 |
| Material thickness of second layer: | 0 |
| Pour Space of material for second layer: | 0 |
| Material thickness of third layer: | 0 |
| Pour Space of material for third layer: | 0 |
| Infiltration On | |
| Infiltration rate: | 2 |
| Infiltration safety factor: | 1 |
| Wetted surface area On | |
| Total Volume Infiltrated (ac-ft.): | 59.408 |
| Total Volume Through Riser (ac-ft.): | 0 |
| Total Volume Through Facility (ac-ft.): | 59.408 |
| Percent Infiltrated: | 100 |
| Total Precip Applied to Facility: | 0 |
| Total Evap From Facility: | 0 |

Discharge Structure

Riser Height: 2 ft.
Riser Diameter: 12 in.

Element Flows To:

Outlet 1 Outlet 2

Gravel Trench Bed Hydraulic Table

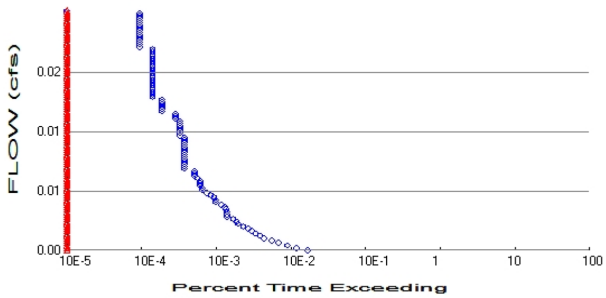
| **Stage(feet)** | **Area(ac.)** | **Volume(ac-ft.)** | **Discharge(cfs)** | **Infilt(cfs)** |
|---|---|---|---|---|
| 0.0000 | 0.023 | 0.000 | 0.000 | 0.000 |
| 0.0278 | 0.023 | 0.000 | 0.000 | 0.047 |
| 0.0556 | 0.023 | 0.000 | 0.000 | 0.048 |
| 0.0833 | 0.024 | 0.000 | 0.000 | 0.048 |
| 0.1111 | 0.024 | 0.001 | 0.000 | 0.049 |
| 0.1389 | 0.025 | 0.001 | 0.000 | 0.050 |
| 0.1667 | 0.025 | 0.001 | 0.000 | 0.051 |
| 0.1944 | 0.025 | 0.001 | 0.000 | 0.052 |
| 0.2222 | 0.026 | 0.002 | 0.000 | 0.053 |
| 0.2500 | 0.026 | 0.002 | 0.000 | 0.054 |
| 0.2778 | 0.027 | 0.002 | 0.000 | 0.054 |
| 0.3056 | 0.027 | 0.003 | 0.000 | 0.055 |
| 0.3333 | 0.028 | 0.003 | 0.000 | 0.056 |
| 0.3611 | 0.028 | 0.003 | 0.000 | 0.057 |
| 0.3889 | 0.029 | 0.004 | 0.000 | 0.058 |
| 0.4167 | 0.029 | 0.004 | 0.000 | 0.059 |
| 0.4444 | 0.029 | 0.004 | 0.000 | 0.060 |
| 0.4722 | 0.030 | 0.005 | 0.000 | 0.061 |
| 0.5000 | 0.030 | 0.005 | 0.000 | 0.062 |
| 0.5278 | 0.031 | 0.005 | 0.000 | 0.062 |
| 0.5556 | 0.031 | 0.006 | 0.000 | 0.063 |
| 0.5833 | 0.032 | 0.006 | 0.000 | 0.064 |
| 0.6111 | 0.032 | 0.006 | 0.000 | 0.065 |

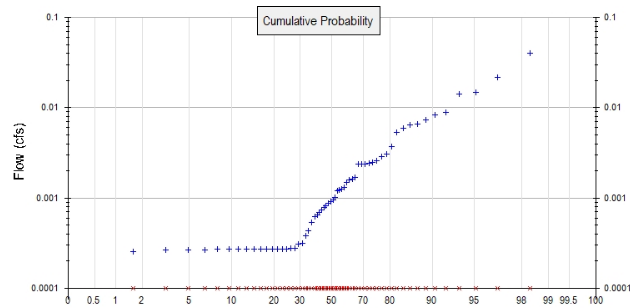
| | | | | |
|---|---|---|---|---|
| 0.6389 | 0.033 | 0.007 | 0.000 | 0.066 |
| 0.6667 | 0.033 | 0.007 | 0.000 | 0.067 |
| 0.6944 | 0.033 | 0.007 | 0.000 | 0.068 |
| 0.7222 | 0.034 | 0.008 | 0.000 | 0.069 |
| 0.7500 | 0.034 | 0.008 | 0.000 | 0.070 |
| 0.7778 | 0.035 | 0.009 | 0.000 | 0.071 |
| 0.8056 | 0.035 | 0.009 | 0.000 | 0.072 |
| 0.8333 | 0.036 | 0.009 | 0.000 | 0.072 |
| 0.8611 | 0.036 | 0.010 | 0.000 | 0.073 |
| 0.8889 | 0.037 | 0.010 | 0.000 | 0.074 |
| 0.9167 | 0.037 | 0.011 | 0.000 | 0.075 |
| 0.9444 | 0.038 | 0.011 | 0.000 | 0.076 |
| 0.9722 | 0.038 | 0.011 | 0.000 | 0.077 |
| 1.0000 | 0.038 | 0.012 | 0.000 | 0.078 |
| 1.0278 | 0.039 | 0.012 | 0.000 | 0.079 |
| 1.0556 | 0.039 | 0.013 | 0.000 | 0.080 |
| 1.0833 | 0.040 | 0.013 | 0.000 | 0.081 |
| 1.1111 | 0.040 | 0.014 | 0.000 | 0.082 |
| 1.1389 | 0.041 | 0.014 | 0.000 | 0.083 |
| 1.1667 | 0.041 | 0.015 | 0.000 | 0.084 |
| 1.1944 | 0.042 | 0.015 | 0.000 | 0.085 |
| 1.2222 | 0.042 | 0.016 | 0.000 | 0.086 |
| 1.2500 | 0.043 | 0.016 | 0.000 | 0.087 |
| 1.2778 | 0.043 | 0.016 | 0.000 | 0.088 |
| 1.3056 | 0.044 | 0.017 | 0.000 | 0.089 |
| 1.3333 | 0.044 | 0.017 | 0.000 | 0.090 |
| 1.3611 | 0.045 | 0.018 | 0.000 | 0.091 |
| 1.3889 | 0.045 | 0.018 | 0.000 | 0.091 |
| 1.4167 | 0.046 | 0.019 | 0.000 | 0.092 |
| 1.4444 | 0.046 | 0.019 | 0.000 | 0.093 |
| 1.4722 | 0.047 | 0.020 | 0.000 | 0.094 |
| 1.5000 | 0.047 | 0.021 | 0.000 | 0.095 |
| 1.5278 | 0.048 | 0.022 | 0.000 | 0.096 |
| 1.5556 | 0.048 | 0.023 | 0.000 | 0.097 |
| 1.5833 | 0.049 | 0.025 | 0.000 | 0.098 |
| 1.6111 | 0.049 | 0.026 | 0.000 | 0.099 |
| 1.6389 | 0.050 | 0.027 | 0.000 | 0.100 |
| 1.6667 | 0.050 | 0.029 | 0.000 | 0.101 |
| 1.6944 | 0.051 | 0.030 | 0.000 | 0.102 |
| 1.7222 | 0.051 | 0.032 | 0.000 | 0.103 |
| 1.7500 | 0.052 | 0.033 | 0.000 | 0.104 |
| 1.7778 | 0.052 | 0.034 | 0.000 | 0.105 |
| 1.8056 | 0.053 | 0.036 | 0.000 | 0.106 |
| 1.8333 | 0.053 | 0.037 | 0.000 | 0.107 |
| 1.8611 | 0.054 | 0.039 | 0.000 | 0.108 |
| 1.8889 | 0.054 | 0.040 | 0.000 | 0.110 |
| 1.9167 | 0.055 | 0.042 | 0.000 | 0.111 |
| 1.9444 | 0.055 | 0.043 | 0.000 | 0.112 |
| 1.9722 | 0.056 | 0.045 | 0.000 | 0.113 |
| 2.0000 | 0.056 | 0.047 | 0.000 | 0.114 |
| 2.0278 | 0.057 | 0.048 | 0.049 | 0.115 |
| 2.0556 | 0.057 | 0.050 | 0.138 | 0.116 |
| 2.0833 | 0.058 | 0.051 | 0.254 | 0.117 |
| 2.1111 | 0.058 | 0.053 | 0.389 | 0.118 |
| 2.1389 | 0.059 | 0.055 | 0.540 | 0.119 |
| 2.1667 | 0.059 | 0.056 | 0.703 | 0.120 |
| 2.1944 | 0.060 | 0.058 | 0.873 | 0.121 |
| 2.2222 | 0.060 | 0.060 | 1.046 | 0.122 |

| | | | | |
|---|---|---|---|---|
| 2.2500 | 0.061 | 0.061 | 1.217 | 0.123 |
| 2.2778 | 0.061 | 0.063 | 1.383 | 0.124 |
| 2.3056 | 0.062 | 0.065 | 1.540 | 0.125 |
| 2.3333 | 0.062 | 0.066 | 1.683 | 0.126 |
| 2.3611 | 0.063 | 0.068 | 1.811 | 0.127 |
| 2.3889 | 0.063 | 0.070 | 1.921 | 0.128 |
| 2.4167 | 0.064 | 0.072 | 2.013 | 0.129 |
| 2.4444 | 0.064 | 0.074 | 2.088 | 0.130 |
| 2.4722 | 0.065 | 0.075 | 2.149 | 0.132 |
| 2.5000 | 0.066 | 0.077 | 2.203 | 0.133 |

# *Analysis Results*

## *POC 1*




+ Predeveloped      x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 0.34
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 0.07
Total Impervious Area: 0.27

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped.  POC #1

| **Return Period** | **Flow(cfs)** |
|---|---|
| 2 year | 0.000989 |
| 5 year | 0.003313 |
| 10 year | 0.00666 |
| 25 year | 0.014769 |
| 50 year | 0.025431 |
| 100 year | 0.042288 |

Flow Frequency Return Periods for Mitigated.  POC #1

| **Return Period** | **Flow(cfs)** |
|---|---|
| 2 year | 0 |
| 5 year | 0 |
| 10 year | 0 |
| 25 year | 0 |
| 50 year | 0 |
| 100 year | 0 |

## Annual Peaks

Annual Peaks for Predeveloped and Mitigated.  POC #1

| **Year** | **Predeveloped** | **Mitigated** |
|---|---|---|
| 1949 | 0.001 | 0.000 |
| 1950 | 0.014 | 0.000 |
| 1951 | 0.002 | 0.000 |
| 1952 | 0.001 | 0.000 |
| 1953 | 0.000 | 0.000 |
| 1954 | 0.002 | 0.000 |
| 1955 | 0.000 | 0.000 |
| 1956 | 0.006 | 0.000 |
| 1957 | 0.001 | 0.000 |
| 1958 | 0.001 | 0.000 |

| | | |
|---|---|---|
| 1959 | 0.001 | 0.000 |
| 1960 | 0.003 | 0.000 |
| 1961 | 0.001 | 0.000 |
| 1962 | 0.000 | 0.000 |
| 1963 | 0.001 | 0.000 |
| 1964 | 0.004 | 0.000 |
| 1965 | 0.001 | 0.000 |
| 1966 | 0.001 | 0.000 |
| 1967 | 0.009 | 0.000 |
| 1968 | 0.002 | 0.000 |
| 1969 | 0.001 | 0.000 |
| 1970 | 0.000 | 0.000 |
| 1971 | 0.001 | 0.000 |
| 1972 | 0.008 | 0.000 |
| 1973 | 0.000 | 0.000 |
| 1974 | 0.001 | 0.000 |
| 1975 | 0.002 | 0.000 |
| 1976 | 0.003 | 0.000 |
| 1977 | 0.000 | 0.000 |
| 1978 | 0.000 | 0.000 |
| 1979 | 0.000 | 0.000 |
| 1980 | 0.000 | 0.000 |
| 1981 | 0.000 | 0.000 |
| 1982 | 0.002 | 0.000 |
| 1983 | 0.000 | 0.000 |
| 1984 | 0.001 | 0.000 |
| 1985 | 0.000 | 0.000 |
| 1986 | 0.000 | 0.000 |
| 1987 | 0.003 | 0.000 |
| 1988 | 0.000 | 0.000 |
| 1989 | 0.000 | 0.000 |
| 1990 | 0.022 | 0.000 |
| 1991 | 0.007 | 0.000 |
| 1992 | 0.000 | 0.000 |
| 1993 | 0.000 | 0.000 |
| 1994 | 0.000 | 0.000 |
| 1995 | 0.005 | 0.000 |
| 1996 | 0.015 | 0.000 |
| 1997 | 0.002 | 0.000 |
| 1998 | 0.001 | 0.000 |
| 1999 | 0.006 | 0.000 |
| 2000 | 0.000 | 0.000 |
| 2001 | 0.000 | 0.000 |
| 2002 | 0.002 | 0.000 |
| 2003 | 0.001 | 0.000 |
| 2004 | 0.001 | 0.000 |
| 2005 | 0.000 | 0.000 |
| 2006 | 0.002 | 0.000 |
| 2007 | 0.040 | 0.000 |
| 2008 | 0.007 | 0.000 |
| 2009 | 0.000 | 0.000 |

## Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated.  POC #1

| Rank | Predeveloped | Mitigated |
|---|---|---|
| 1 | 0.0404 | 0.0000 |
| 2 | 0.0217 | 0.0000 |
| 3 | 0.0149 | 0.0000 |

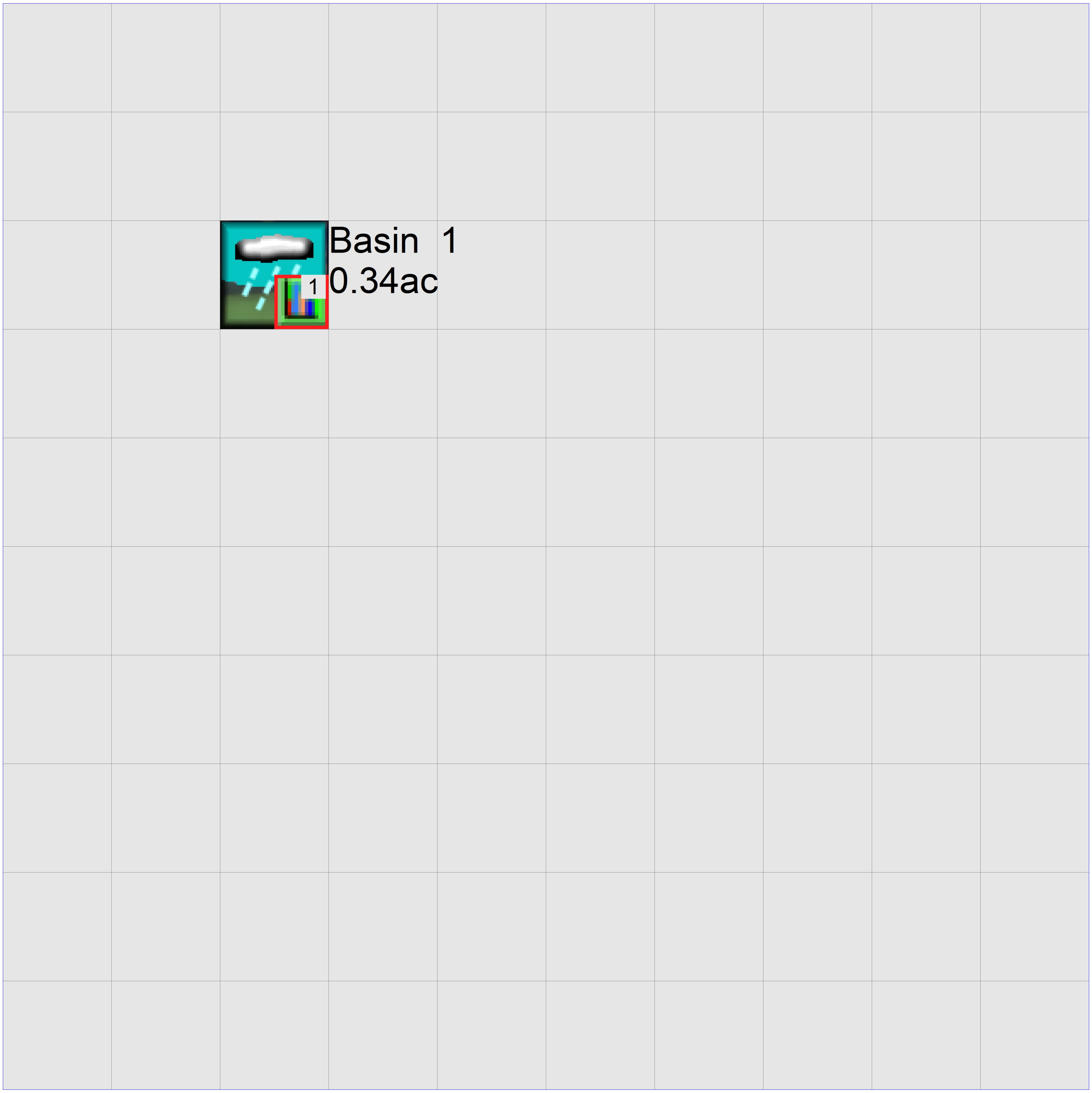
| | | |
|---|---|---|
| 4 | 0.0143 | 0.0000 |
| 5 | 0.0089 | 0.0000 |
| 6 | 0.0083 | 0.0000 |
| 7 | 0.0073 | 0.0000 |
| 8 | 0.0066 | 0.0000 |
| 9 | 0.0064 | 0.0000 |
| 10 | 0.0059 | 0.0000 |
| 11 | 0.0053 | 0.0000 |
| 12 | 0.0037 | 0.0000 |
| 13 | 0.0031 | 0.0000 |
| 14 | 0.0029 | 0.0000 |
| 15 | 0.0026 | 0.0000 |
| 16 | 0.0025 | 0.0000 |
| 17 | 0.0024 | 0.0000 |
| 18 | 0.0024 | 0.0000 |
| 19 | 0.0024 | 0.0000 |
| 20 | 0.0024 | 0.0000 |
| 21 | 0.0017 | 0.0000 |
| 22 | 0.0016 | 0.0000 |
| 23 | 0.0016 | 0.0000 |
| 24 | 0.0015 | 0.0000 |
| 25 | 0.0013 | 0.0000 |
| 26 | 0.0013 | 0.0000 |
| 27 | 0.0012 | 0.0000 |
| 28 | 0.0012 | 0.0000 |
| 29 | 0.0010 | 0.0000 |
| 30 | 0.0009 | 0.0000 |
| 31 | 0.0009 | 0.0000 |
| 32 | 0.0009 | 0.0000 |
| 33 | 0.0008 | 0.0000 |
| 34 | 0.0008 | 0.0000 |
| 35 | 0.0007 | 0.0000 |
| 36 | 0.0007 | 0.0000 |
| 37 | 0.0006 | 0.0000 |
| 38 | 0.0006 | 0.0000 |
| 39 | 0.0005 | 0.0000 |
| 40 | 0.0004 | 0.0000 |
| 41 | 0.0004 | 0.0000 |
| 42 | 0.0003 | 0.0000 |
| 43 | 0.0003 | 0.0000 |
| 44 | 0.0003 | 0.0000 |
| 45 | 0.0003 | 0.0000 |
| 46 | 0.0003 | 0.0000 |
| 47 | 0.0003 | 0.0000 |
| 48 | 0.0003 | 0.0000 |
| 49 | 0.0003 | 0.0000 |
| 50 | 0.0003 | 0.0000 |
| 51 | 0.0003 | 0.0000 |
| 52 | 0.0003 | 0.0000 |
| 53 | 0.0003 | 0.0000 |
| 54 | 0.0003 | 0.0000 |
| 55 | 0.0003 | 0.0000 |
| 56 | 0.0003 | 0.0000 |
| 57 | 0.0003 | 0.0000 |
| 58 | 0.0003 | 0.0000 |
| 59 | 0.0003 | 0.0000 |
| 60 | 0.0003 | 0.0000 |
| 61 | 0.0002 | 0.0000 |

## Duration Flows
The Facility PASSED

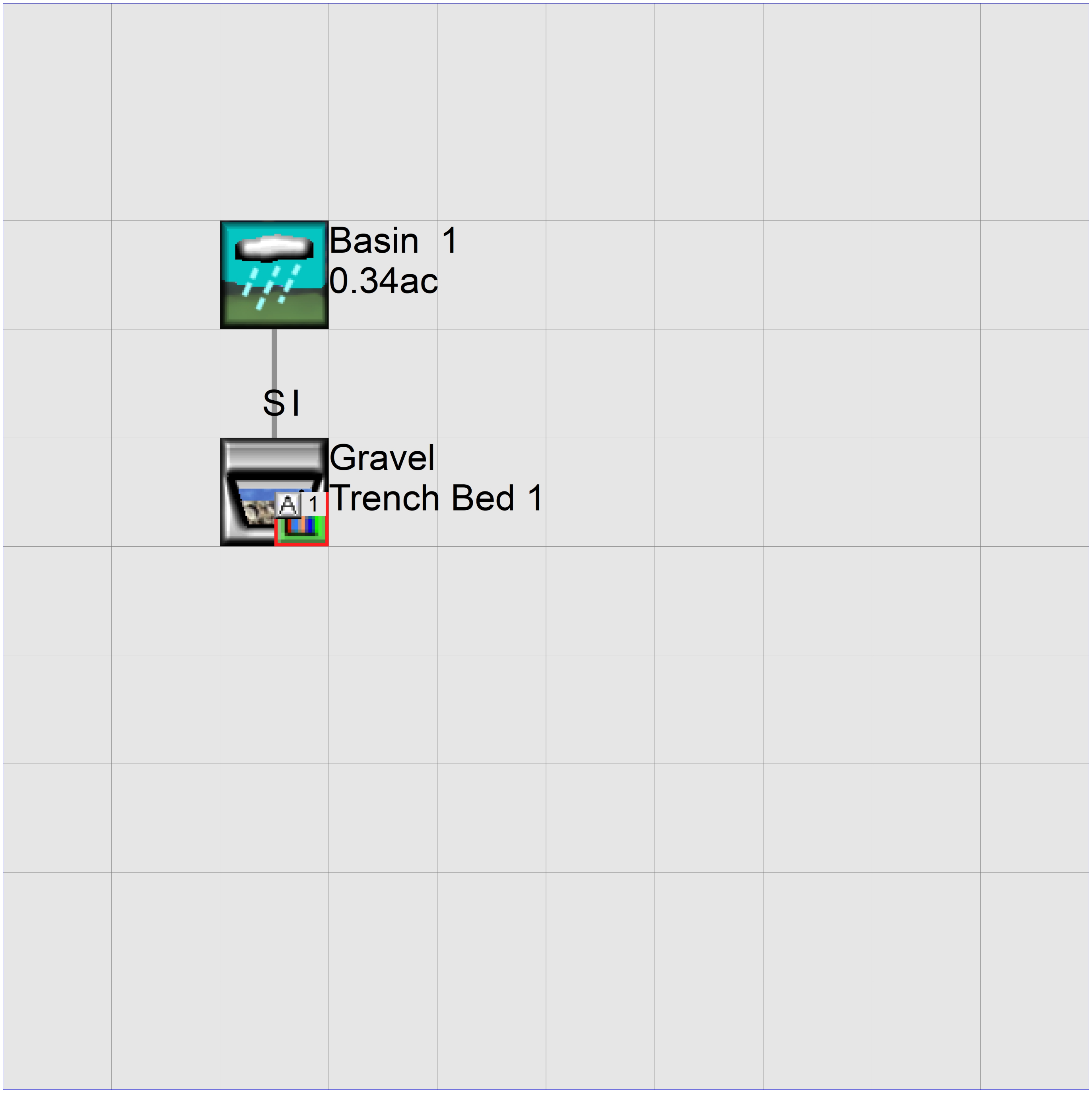
| Flow(cfs) | Predev | Mit | Percentage | Pass/Fail |
|---|---|---|---|---|
| 0.0005 | 361 | 0 | 0 | Pass |
| 0.0007 | 255 | 0 | 0 | Pass |
| 0.0010 | 193 | 0 | 0 | Pass |
| 0.0013 | 144 | 0 | 0 | Pass |
| 0.0015 | 119 | 0 | 0 | Pass |
| 0.0018 | 93 | 0 | 0 | Pass |
| 0.0020 | 82 | 0 | 0 | Pass |
| 0.0023 | 72 | 0 | 0 | Pass |
| 0.0025 | 64 | 0 | 0 | Pass |
| 0.0028 | 57 | 0 | 0 | Pass |
| 0.0030 | 49 | 0 | 0 | Pass |
| 0.0033 | 42 | 0 | 0 | Pass |
| 0.0035 | 40 | 0 | 0 | Pass |
| 0.0038 | 36 | 0 | 0 | Pass |
| 0.0040 | 30 | 0 | 0 | Pass |
| 0.0043 | 30 | 0 | 0 | Pass |
| 0.0045 | 30 | 0 | 0 | Pass |
| 0.0048 | 29 | 0 | 0 | Pass |
| 0.0050 | 28 | 0 | 0 | Pass |
| 0.0053 | 25 | 0 | 0 | Pass |
| 0.0055 | 21 | 0 | 0 | Pass |
| 0.0058 | 21 | 0 | 0 | Pass |
| 0.0060 | 20 | 0 | 0 | Pass |
| 0.0063 | 18 | 0 | 0 | Pass |
| 0.0065 | 16 | 0 | 0 | Pass |
| 0.0068 | 14 | 0 | 0 | Pass |
| 0.0070 | 14 | 0 | 0 | Pass |
| 0.0073 | 13 | 0 | 0 | Pass |
| 0.0075 | 13 | 0 | 0 | Pass |
| 0.0078 | 13 | 0 | 0 | Pass |
| 0.0081 | 12 | 0 | 0 | Pass |
| 0.0083 | 11 | 0 | 0 | Pass |
| 0.0086 | 11 | 0 | 0 | Pass |
| 0.0088 | 11 | 0 | 0 | Pass |
| 0.0091 | 8 | 0 | 0 | Pass |
| 0.0093 | 8 | 0 | 0 | Pass |
| 0.0096 | 8 | 0 | 0 | Pass |
| 0.0098 | 8 | 0 | 0 | Pass |
| 0.0101 | 8 | 0 | 0 | Pass |
| 0.0103 | 8 | 0 | 0 | Pass |
| 0.0106 | 8 | 0 | 0 | Pass |
| 0.0108 | 8 | 0 | 0 | Pass |
| 0.0111 | 8 | 0 | 0 | Pass |
| 0.0113 | 8 | 0 | 0 | Pass |
| 0.0116 | 8 | 0 | 0 | Pass |
| 0.0118 | 8 | 0 | 0 | Pass |
| 0.0121 | 8 | 0 | 0 | Pass |
| 0.0123 | 8 | 0 | 0 | Pass |
| 0.0126 | 7 | 0 | 0 | Pass |
| 0.0128 | 7 | 0 | 0 | Pass |
| 0.0131 | 7 | 0 | 0 | Pass |
| 0.0133 | 7 | 0 | 0 | Pass |
| 0.0136 | 7 | 0 | 0 | Pass |

| | | | | |
|---|---|---|---|---|
| 0.0138 | 7 | 0 | 0 | Pass |
| 0.0141 | 7 | 0 | 0 | Pass |
| 0.0143 | 6 | 0 | 0 | Pass |
| 0.0146 | 6 | 0 | 0 | Pass |
| 0.0149 | 6 | 0 | 0 | Pass |
| 0.0151 | 4 | 0 | 0 | Pass |
| 0.0154 | 4 | 0 | 0 | Pass |
| 0.0156 | 4 | 0 | 0 | Pass |
| 0.0159 | 4 | 0 | 0 | Pass |
| 0.0161 | 4 | 0 | 0 | Pass |
| 0.0164 | 4 | 0 | 0 | Pass |
| 0.0166 | 3 | 0 | 0 | Pass |
| 0.0169 | 3 | 0 | 0 | Pass |
| 0.0171 | 3 | 0 | 0 | Pass |
| 0.0174 | 3 | 0 | 0 | Pass |
| 0.0176 | 3 | 0 | 0 | Pass |
| 0.0179 | 3 | 0 | 0 | Pass |
| 0.0181 | 3 | 0 | 0 | Pass |
| 0.0184 | 3 | 0 | 0 | Pass |
| 0.0186 | 3 | 0 | 0 | Pass |
| 0.0189 | 3 | 0 | 0 | Pass |
| 0.0191 | 3 | 0 | 0 | Pass |
| 0.0194 | 3 | 0 | 0 | Pass |
| 0.0196 | 3 | 0 | 0 | Pass |
| 0.0199 | 3 | 0 | 0 | Pass |
| 0.0201 | 3 | 0 | 0 | Pass |
| 0.0204 | 3 | 0 | 0 | Pass |
| 0.0206 | 3 | 0 | 0 | Pass |
| 0.0209 | 3 | 0 | 0 | Pass |
| 0.0211 | 3 | 0 | 0 | Pass |
| 0.0214 | 3 | 0 | 0 | Pass |
| 0.0217 | 3 | 0 | 0 | Pass |
| 0.0219 | 2 | 0 | 0 | Pass |
| 0.0222 | 2 | 0 | 0 | Pass |
| 0.0224 | 2 | 0 | 0 | Pass |
| 0.0227 | 2 | 0 | 0 | Pass |
| 0.0229 | 2 | 0 | 0 | Pass |
| 0.0232 | 2 | 0 | 0 | Pass |
| 0.0234 | 2 | 0 | 0 | Pass |
| 0.0237 | 2 | 0 | 0 | Pass |
| 0.0239 | 2 | 0 | 0 | Pass |
| 0.0242 | 2 | 0 | 0 | Pass |
| 0.0244 | 2 | 0 | 0 | Pass |
| 0.0247 | 2 | 0 | 0 | Pass |
| 0.0249 | 2 | 0 | 0 | Pass |
| 0.0252 | 2 | 0 | 0 | Pass |
| 0.0254 | 2 | 0 | 0 | Pass |

# Appendix
## Predeveloped Schematic



*Mitigated Schematic*



# Disclaimer

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Clear Creek Solutions, Inc.
6200 Capitol Blvd. Ste F
Olympia, WA. 98501
Toll Free 1(866)943-0304
Local (360)943-0304

www.clearcreeksolutions.com

# WWHM2012

# PROJECT REPORT

## BASIN 1:
## TREATMENT

# General Model Information

| | |
|---|---|
| Project Name: | 1883.01 Issaquah Basin 1 Treatment |
| Site Name: | |
| Site Address: | |
| City: | |
| Report Date: | 6/17/2019 |
| Gage: | Seatac |
| Data Start: | 1948/10/01 |
| Data End: | 2009/09/30 |
| Timestep: | 15 Minute |
| Precip Scale: | 1.333 |
| Version Date: | 2018/10/10 |
| Version: | 4.2.16 |

## POC Thresholds

| | |
|---|---|
| Low Flow Threshold for POC1: | 50 Percent of the 2 Year |
| High Flow Threshold for POC1: | 50 Year |

## *Landuse Basin Data*
### *Predeveloped Land Use*

### Basin 1

| | |
|---|---|
| Bypass: | No |
| GroundWater: | No |
| Pervious Land Use | acre |
| A B, Forest, Flat | 1.25 |
| Pervious Total | 1.25 |
| Impervious Land Use | acre |
| Impervious Total | 0 |
| Basin Total | 1.25 |

Element Flows To:

| Surface | Interflow | Groundwater |
|---|---|---|

## *Mitigated Land Use*

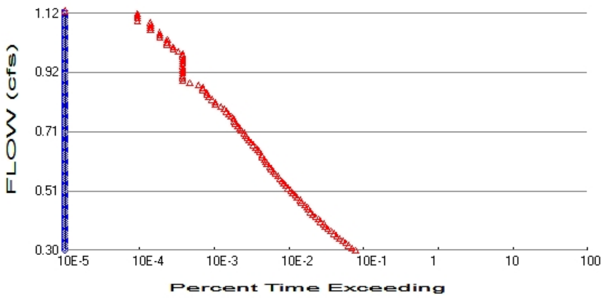
### Basin 1

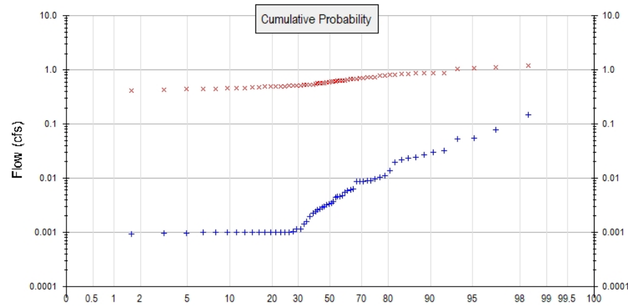
| | |
|---|---|
| Bypass: | No |
| GroundWater: | No |
| Pervious Land Use | acre |
| A B, Lawn, Flat | 0.12 |
| Pervious Total | 0.12 |
| Impervious Land Use | acre |
| SIDEWALKS FLAT | 0.08 |
| PARKING FLAT | 1.05 |
| Impervious Total | 1.13 |
| Basin Total | 1.25 |

Element Flows To:

| Surface | Interflow | Groundwater |
|---|---|---|

# *Analysis Results*
## *POC 1*




+ Predeveloped x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 1.25
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 0.12
Total Impervious Area: 1.13

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

| **Return Period** | **Flow(cfs)** |
|---|---|
| 2 year | 0.003635 |
| 5 year | 0.012179 |
| 10 year | 0.024484 |
| 25 year | 0.054298 |
| 50 year | 0.093497 |
| 100 year | 0.155471 |

Flow Frequency Return Periods for Mitigated. POC #1

| **Return Period** | **Flow(cfs)** |
|---|---|
| 2 year | 0.607447 |
| 5 year | 0.767247 |
| 10 year | 0.875817 |
| 25 year | 1.016646 |
| 50 year | 1.12443 |
| 100 year | 1.234829 |

## Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

| **Year** | **Predeveloped** | **Mitigated** |
|---|---|---|
| 1949 | 0.003 | 0.796 |
| 1950 | 0.052 | 0.832 |
| 1951 | 0.009 | 0.492 |
| 1952 | 0.003 | 0.415 |
| 1953 | 0.001 | 0.466 |
| 1954 | 0.009 | 0.495 |
| 1955 | 0.001 | 0.566 |
| 1956 | 0.022 | 0.535 |
| 1957 | 0.003 | 0.614 |
| 1958 | 0.004 | 0.500 |

| | | |
|---|---|---|
| 1959 | 0.003 | 0.523 |
| 1960 | 0.011 | 0.511 |
| 1961 | 0.005 | 0.515 |
| 1962 | 0.001 | 0.453 |
| 1963 | 0.005 | 0.527 |
| 1964 | 0.014 | 0.510 |
| 1965 | 0.003 | 0.638 |
| 1966 | 0.002 | 0.416 |
| 1967 | 0.033 | 0.723 |
| 1968 | 0.009 | 0.858 |
| 1969 | 0.004 | 0.567 |
| 1970 | 0.001 | 0.564 |
| 1971 | 0.005 | 0.669 |
| 1972 | 0.031 | 0.698 |
| 1973 | 0.001 | 0.430 |
| 1974 | 0.003 | 0.630 |
| 1975 | 0.006 | 0.692 |
| 1976 | 0.010 | 0.486 |
| 1977 | 0.001 | 0.511 |
| 1978 | 0.001 | 0.672 |
| 1979 | 0.001 | 0.871 |
| 1980 | 0.001 | 0.803 |
| 1981 | 0.001 | 0.614 |
| 1982 | 0.006 | 0.872 |
| 1983 | 0.001 | 0.711 |
| 1984 | 0.002 | 0.444 |
| 1985 | 0.001 | 0.605 |
| 1986 | 0.001 | 0.531 |
| 1987 | 0.011 | 0.823 |
| 1988 | 0.001 | 0.500 |
| 1989 | 0.001 | 0.718 |
| 1990 | 0.080 | 1.095 |
| 1991 | 0.027 | 0.886 |
| 1992 | 0.001 | 0.440 |
| 1993 | 0.001 | 0.474 |
| 1994 | 0.001 | 0.446 |
| 1995 | 0.019 | 0.549 |
| 1996 | 0.055 | 0.626 |
| 1997 | 0.009 | 0.579 |
| 1998 | 0.005 | 0.580 |
| 1999 | 0.024 | 1.198 |
| 2000 | 0.001 | 0.596 |
| 2001 | 0.001 | 0.667 |
| 2002 | 0.006 | 0.736 |
| 2003 | 0.002 | 0.640 |
| 2004 | 0.003 | 1.128 |
| 2005 | 0.001 | 0.503 |
| 2006 | 0.009 | 0.457 |
| 2007 | 0.148 | 1.060 |
| 2008 | 0.024 | 0.856 |
| 2009 | 0.002 | 0.789 |

## Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

| **Rank** | **Predeveloped** | **Mitigated** |
|---|---|---|
| 1 | 0.1484 | 1.1979 |
| 2 | 0.0796 | 1.1278 |
| 3 | 0.0548 | 1.0950 |

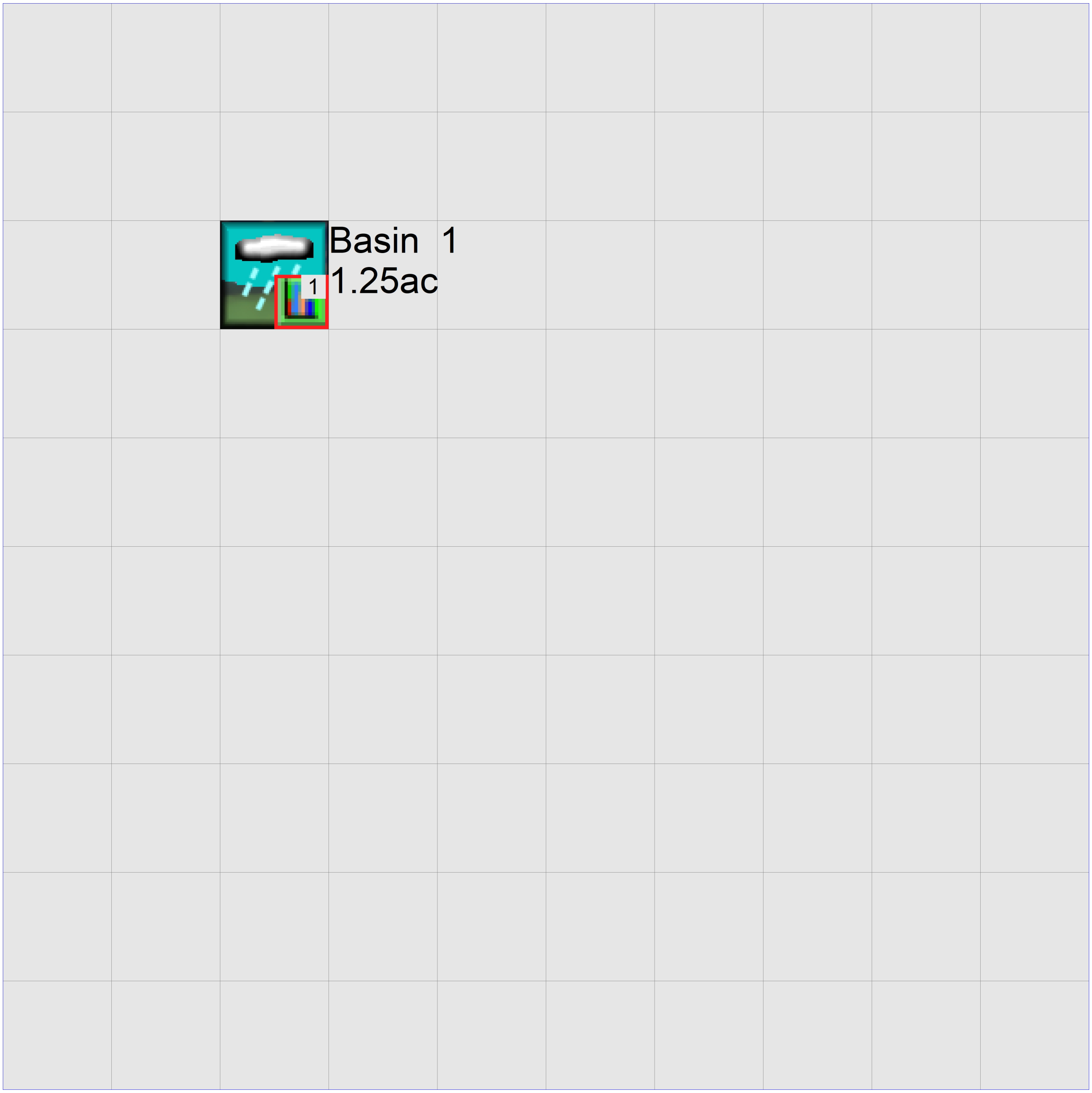
| | | |
|---|---|---|
| 4 | 0.0525 | 1.0599 |
| 5 | 0.0326 | 0.8860 |
| 6 | 0.0305 | 0.8721 |
| 7 | 0.0268 | 0.8706 |
| 8 | 0.0242 | 0.8582 |
| 9 | 0.0236 | 0.8555 |
| 10 | 0.0217 | 0.8317 |
| 11 | 0.0194 | 0.8227 |
| 12 | 0.0136 | 0.8027 |
| 13 | 0.0113 | 0.7957 |
| 14 | 0.0105 | 0.7891 |
| 15 | 0.0096 | 0.7357 |
| 16 | 0.0091 | 0.7235 |
| 17 | 0.0089 | 0.7179 |
| 18 | 0.0087 | 0.7109 |
| 19 | 0.0087 | 0.6982 |
| 20 | 0.0087 | 0.6923 |
| 21 | 0.0062 | 0.6719 |
| 22 | 0.0060 | 0.6691 |
| 23 | 0.0059 | 0.6671 |
| 24 | 0.0055 | 0.6396 |
| 25 | 0.0048 | 0.6379 |
| 26 | 0.0046 | 0.6298 |
| 27 | 0.0046 | 0.6264 |
| 28 | 0.0044 | 0.6143 |
| 29 | 0.0037 | 0.6142 |
| 30 | 0.0035 | 0.6048 |
| 31 | 0.0034 | 0.5955 |
| 32 | 0.0032 | 0.5803 |
| 33 | 0.0030 | 0.5786 |
| 34 | 0.0029 | 0.5671 |
| 35 | 0.0027 | 0.5657 |
| 36 | 0.0026 | 0.5643 |
| 37 | 0.0024 | 0.5495 |
| 38 | 0.0023 | 0.5353 |
| 39 | 0.0020 | 0.5305 |
| 40 | 0.0016 | 0.5270 |
| 41 | 0.0014 | 0.5234 |
| 42 | 0.0012 | 0.5148 |
| 43 | 0.0011 | 0.5111 |
| 44 | 0.0010 | 0.5107 |
| 45 | 0.0010 | 0.5098 |
| 46 | 0.0010 | 0.5029 |
| 47 | 0.0010 | 0.5001 |
| 48 | 0.0010 | 0.4998 |
| 49 | 0.0010 | 0.4952 |
| 50 | 0.0010 | 0.4923 |
| 51 | 0.0010 | 0.4856 |
| 52 | 0.0010 | 0.4739 |
| 53 | 0.0010 | 0.4664 |
| 54 | 0.0010 | 0.4568 |
| 55 | 0.0010 | 0.4534 |
| 56 | 0.0010 | 0.4456 |
| 57 | 0.0010 | 0.4441 |
| 58 | 0.0010 | 0.4404 |
| 59 | 0.0010 | 0.4305 |
| 60 | 0.0009 | 0.4163 |
| 61 | 0.0007 | 0.4150 |

## Water Quality

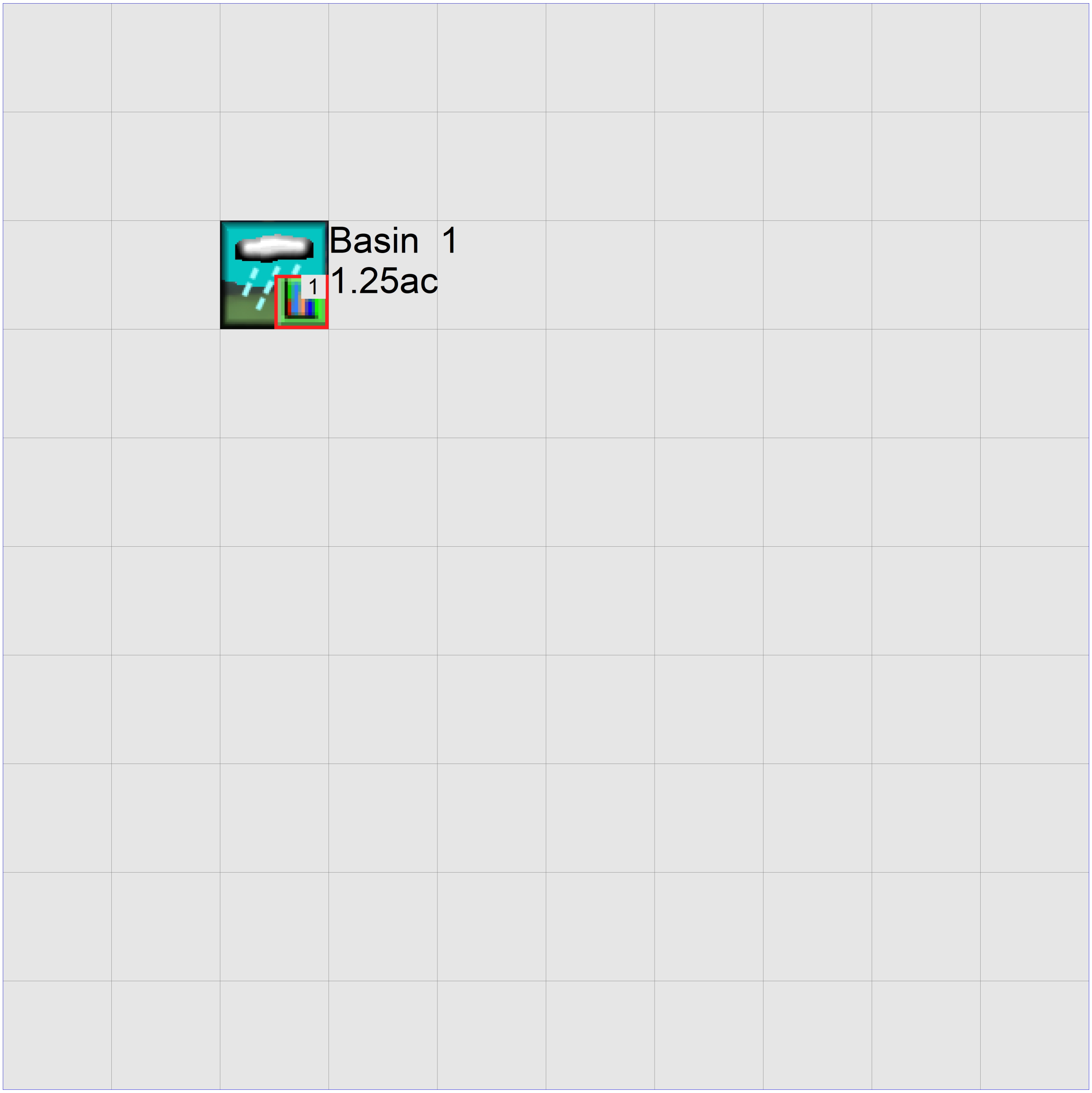
Water Quality BMP Flow and Volume for POC #1

| | |
|---|---|
| On-line facility volume: | 0.1817 acre-feet |
| On-line facility target flow: | 0.2498 cfs. |
| Adjusted for 15 min: | 0.2498 cfs. |
| Off-line facility target flow: | 0.1409 cfs. |
| Adjusted for 15 min: | 0.1409 cfs. |

# Appendix
## *Predeveloped Schematic*



## *Mitigated Schematic*



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Clear Creek Solutions, Inc.
6200 Capitol Blvd. Ste F
Olympia, WA. 98501
Toll Free 1(866)943-0304
Local (360)943-0304

www.clearcreeksolutions.com

# WWHM2012

# PROJECT REPORT

## BASIN 2: TREATMENT

## *General Model Information*

| | |
|---|---|
| Project Name: | 1883.01 Issaquah Basin 2 Treatment |
| Site Name: | |
| Site Address: | |
| City: | |
| Report Date: | 6/17/2019 |
| Gage: | Seatac |
| Data Start: | 1948/10/01 |
| Data End: | 2009/09/30 |
| Timestep: | 15 Minute |
| Precip Scale: | 1.333 |
| Version Date: | 2018/10/10 |
| Version: | 4.2.16 |

### *POC Thresholds*

| | |
|---|---|
| Low Flow Threshold for POC1: | 50 Percent of the 2 Year |
| High Flow Threshold for POC1: | 50 Year |

# *Landuse Basin Data*
## *Predeveloped Land Use*

### Basin 1

| | |
|---|---|
| Bypass: | No |
| GroundWater: | No |
| Pervious Land Use | acre |
| A B, Forest, Flat | 1.85 |
| Pervious Total | 1.85 |
| Impervious Land Use | acre |
| Impervious Total | 0 |
| Basin Total | 1.85 |

Element Flows To:

| Surface | Interflow | Groundwater |
|---|---|---|

## *Mitigated Land Use*

## Basin 1

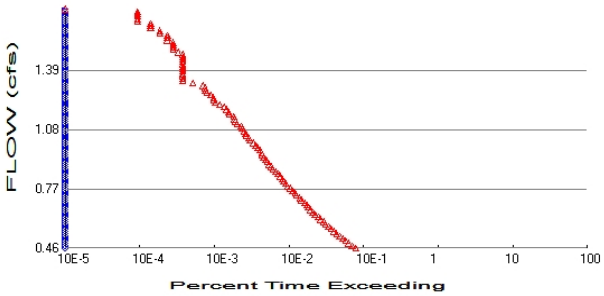
| | |
|---|---|
| Bypass: | No |
| GroundWater: | No |
| Pervious Land Use | acre |
| A B, Lawn, Flat | 0.13 |
| Pervious Total | 0.13 |
| Impervious Land Use | acre |
| ROOF TOPS FLAT | 1 |
| SIDEWALKS FLAT | 0.07 |
| PARKING FLAT | 0.65 |
| Impervious Total | 1.72 |
| Basin Total | 1.85 |

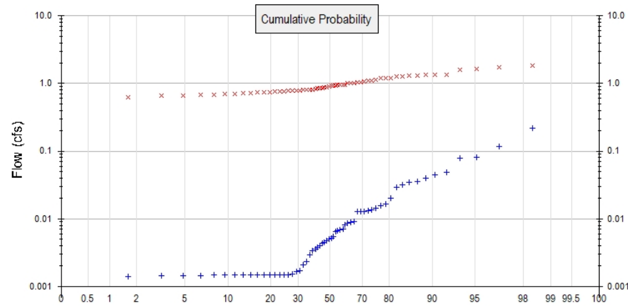
Element Flows To:

| Surface | Interflow | Groundwater |
|---|---|---|

# Analysis Results

## POC 1




+ Predeveloped x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 1.85
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 0.13
Total Impervious Area: 1.72

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

| **Return Period** | **Flow(cfs)** |
|---|---|
| 2 year | 0.00538 |
| 5 year | 0.018025 |
| 10 year | 0.036237 |
| 25 year | 0.080361 |
| 50 year | 0.138375 |
| 100 year | 0.230097 |

Flow Frequency Return Periods for Mitigated. POC #1

| **Return Period** | **Flow(cfs)** |
|---|---|
| 2 year | 0.921839 |
| 5 year | 1.16372 |
| 10 year | 1.327989 |
| 25 year | 1.540995 |
| 50 year | 1.703974 |
| 100 year | 1.870871 |

## Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

| **Year** | **Predeveloped** | **Mitigated** |
|---|---|---|
| 1949 | 0.004 | 1.202 |
| 1950 | 0.078 | 1.266 |
| 1951 | 0.013 | 0.742 |
| 1952 | 0.004 | 0.632 |
| 1953 | 0.001 | 0.710 |
| 1954 | 0.013 | 0.749 |
| 1955 | 0.002 | 0.861 |
| 1956 | 0.032 | 0.812 |
| 1957 | 0.005 | 0.930 |
| 1958 | 0.006 | 0.760 |

| | | |
|---|---|---|
| 1959 | 0.005 | 0.797 |
| 1960 | 0.017 | 0.774 |
| 1961 | 0.008 | 0.780 |
| 1962 | 0.001 | 0.690 |
| 1963 | 0.007 | 0.797 |
| 1964 | 0.020 | 0.776 |
| 1965 | 0.004 | 0.963 |
| 1966 | 0.003 | 0.632 |
| 1967 | 0.048 | 1.101 |
| 1968 | 0.013 | 1.306 |
| 1969 | 0.007 | 0.863 |
| 1970 | 0.001 | 0.856 |
| 1971 | 0.007 | 1.018 |
| 1972 | 0.045 | 1.050 |
| 1973 | 0.001 | 0.655 |
| 1974 | 0.005 | 0.956 |
| 1975 | 0.009 | 1.054 |
| 1976 | 0.014 | 0.736 |
| 1977 | 0.001 | 0.778 |
| 1978 | 0.002 | 1.023 |
| 1979 | 0.001 | 1.325 |
| 1980 | 0.001 | 1.212 |
| 1981 | 0.001 | 0.934 |
| 1982 | 0.009 | 1.325 |
| 1983 | 0.002 | 1.082 |
| 1984 | 0.004 | 0.674 |
| 1985 | 0.001 | 0.921 |
| 1986 | 0.002 | 0.808 |
| 1987 | 0.016 | 1.252 |
| 1988 | 0.001 | 0.761 |
| 1989 | 0.001 | 1.093 |
| 1990 | 0.118 | 1.642 |
| 1991 | 0.040 | 1.332 |
| 1992 | 0.001 | 0.670 |
| 1993 | 0.001 | 0.721 |
| 1994 | 0.001 | 0.678 |
| 1995 | 0.029 | 0.836 |
| 1996 | 0.081 | 0.945 |
| 1997 | 0.013 | 0.873 |
| 1998 | 0.007 | 0.883 |
| 1999 | 0.035 | 1.823 |
| 2000 | 0.001 | 0.901 |
| 2001 | 0.001 | 1.015 |
| 2002 | 0.009 | 1.119 |
| 2003 | 0.003 | 0.966 |
| 2004 | 0.004 | 1.715 |
| 2005 | 0.001 | 0.761 |
| 2006 | 0.013 | 0.689 |
| 2007 | 0.220 | 1.611 |
| 2008 | 0.036 | 1.287 |
| 2009 | 0.002 | 1.201 |

## Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated.  POC #1

| Rank | Predeveloped | Mitigated |
|---|---|---|
| 1 | 0.2197 | 1.8230 |
| 2 | 0.1178 | 1.7155 |
| 3 | 0.0812 | 1.6415 |

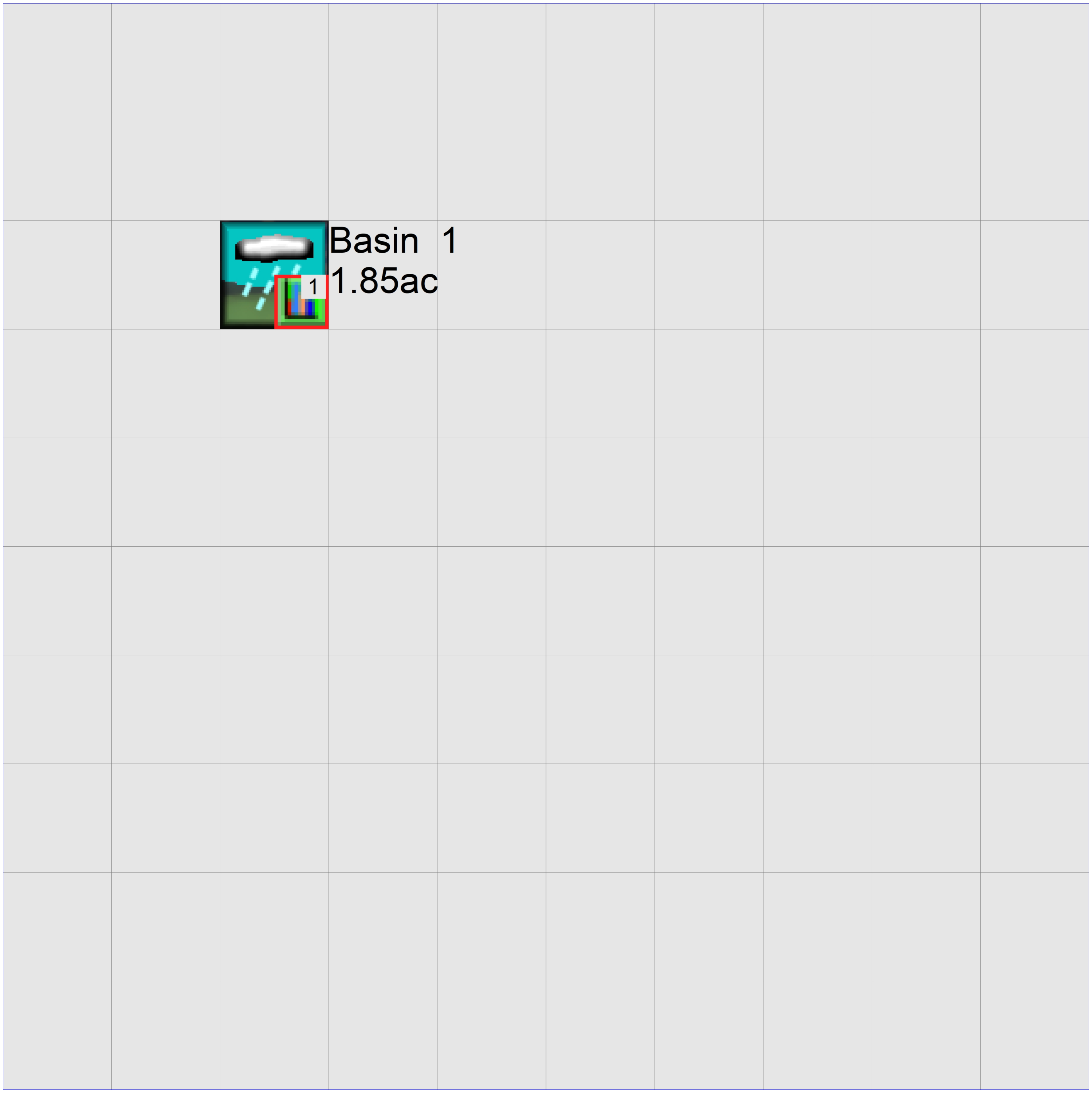
| | | |
|---|---|---|
| 4 | 0.0777 | 1.6109 |
| 5 | 0.0483 | 1.3320 |
| 6 | 0.0452 | 1.3252 |
| 7 | 0.0396 | 1.3251 |
| 8 | 0.0358 | 1.3062 |
| 9 | 0.0350 | 1.2866 |
| 10 | 0.0321 | 1.2659 |
| 11 | 0.0288 | 1.2523 |
| 12 | 0.0201 | 1.2120 |
| 13 | 0.0167 | 1.2020 |
| 14 | 0.0156 | 1.2012 |
| 15 | 0.0142 | 1.1190 |
| 16 | 0.0134 | 1.1012 |
| 17 | 0.0132 | 1.0927 |
| 18 | 0.0129 | 1.0820 |
| 19 | 0.0128 | 1.0537 |
| 20 | 0.0128 | 1.0503 |
| 21 | 0.0092 | 1.0228 |
| 22 | 0.0088 | 1.0178 |
| 23 | 0.0087 | 1.0154 |
| 24 | 0.0081 | 0.9660 |
| 25 | 0.0070 | 0.9633 |
| 26 | 0.0068 | 0.9557 |
| 27 | 0.0067 | 0.9452 |
| 28 | 0.0065 | 0.9342 |
| 29 | 0.0055 | 0.9298 |
| 30 | 0.0052 | 0.9206 |
| 31 | 0.0050 | 0.9010 |
| 32 | 0.0047 | 0.8832 |
| 33 | 0.0044 | 0.8731 |
| 34 | 0.0043 | 0.8631 |
| 35 | 0.0040 | 0.8611 |
| 36 | 0.0038 | 0.8559 |
| 37 | 0.0035 | 0.8362 |
| 38 | 0.0034 | 0.8123 |
| 39 | 0.0029 | 0.8075 |
| 40 | 0.0023 | 0.7971 |
| 41 | 0.0021 | 0.7967 |
| 42 | 0.0017 | 0.7801 |
| 43 | 0.0017 | 0.7779 |
| 44 | 0.0015 | 0.7759 |
| 45 | 0.0015 | 0.7739 |
| 46 | 0.0015 | 0.7608 |
| 47 | 0.0015 | 0.7607 |
| 48 | 0.0015 | 0.7602 |
| 49 | 0.0015 | 0.7494 |
| 50 | 0.0015 | 0.7423 |
| 51 | 0.0015 | 0.7358 |
| 52 | 0.0015 | 0.7212 |
| 53 | 0.0015 | 0.7099 |
| 54 | 0.0015 | 0.6901 |
| 55 | 0.0015 | 0.6895 |
| 56 | 0.0015 | 0.6783 |
| 57 | 0.0015 | 0.6743 |
| 58 | 0.0014 | 0.6700 |
| 59 | 0.0014 | 0.6552 |
| 60 | 0.0014 | 0.6318 |
| 61 | 0.0010 | 0.6316 |

## Water Quality

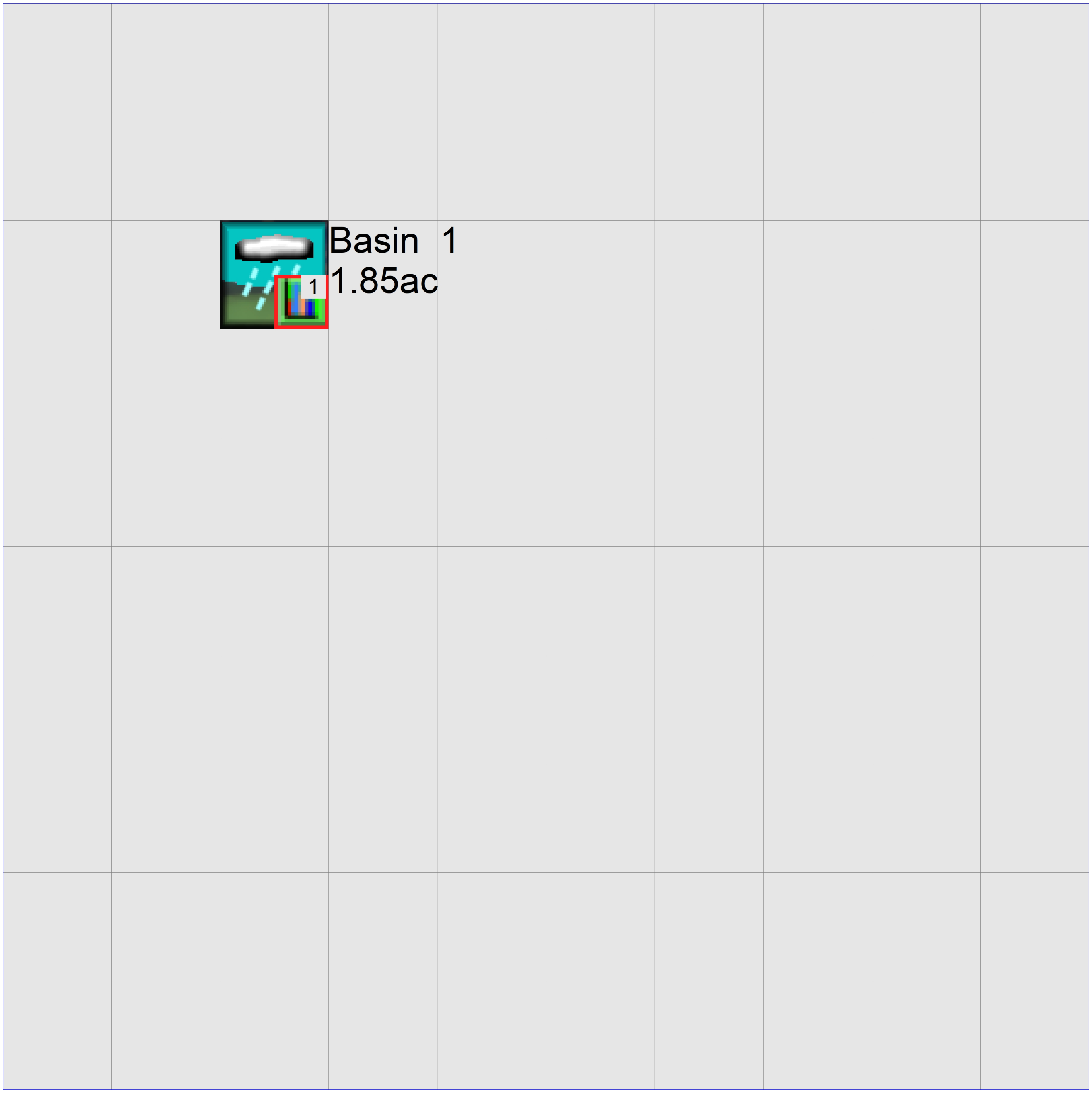
Water Quality BMP Flow and Volume for POC #1

| | |
|---|---|
| On-line facility volume: | 0.2767 acre-feet |
| On-line facility target flow: | 0.3799 cfs. |
| Adjusted for 15 min: | 0.3799 cfs. |
| Off-line facility target flow: | 0.2142 cfs. |
| Adjusted for 15 min: | 0.2142 cfs. |

# Appendix
## Predeveloped Schematic



*Mitigated Schematic*



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Clear Creek Solutions, Inc.
6200 Capitol Blvd. Ste F
Olympia, WA. 98501
Toll Free 1(866)943-0304
Local (360)943-0304

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